FIRE SEVERITY MEASURES AND ENGINEERING DEMAND PARAMETERS FOR PROBABILISTIC STRUCTURAL FIRE ENGINEERING

by

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A thesis submitted in partial fulfilment
of the requirements for the degree of
Doctor of Philosophy

In

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ABSTRACT

Probabilistic Structural Fire Engineering (PSFE) has been introduced to overcome the limitations of current conventional approaches used for the design of fire-exposed structures. Current structural fire design investigates worst-case fire scenarios and include multiple thermal and structural analyses. PSFE permits buildings to be designed to a level of life safety or economic loss that may occur in future fire events with the help of a probabilistic approach. This thesis presents modifications to the adoption of a Performance-Based Earthquake Engineering (PBEE) framework in Probabilistic Structural Fire Engineering (PSFE).

The probabilistic approach runs through a series of interrelationships between different variables, and successive convolution integrals of these interrelationships result in probabilities of different measures. The process starts with the definition of a fire severity measure (FSM), which best relates fire hazard intensity with structural response. It is identified by satisfying efficiency and sufficiency criteria as described by the PBEE framework. The relationship between a fire hazard and corresponding structural response is established by analysis methods. One method that has been used to quantify this relationship in PSFE is Incremental Fire Analysis (IFA). The existing IFA approach produces unrealistic fire scenarios, as fire profiles may be scaled to wide ranges of fire severity levels, which may not physically represent any real fires. Two new techniques are introduced in this thesis to limit extensive scaling.

In order to obtain an annual rate of exceedance of fire hazard and structural response for an office building, an occurrence model and an attenuation model for office fires are generated for both Christchurch city and New Zealand. The results show that Christchurch city is 15% less likely to experience fires that have the potential to cause structural failures in comparison to all of New Zealand.

In establishing better predictive relationships between fires and structural response, cumulative incident radiation (a fire hazard property) is found to be the most appropriate fire severity measure.

This research brings together existing research on various sources of uncertainty in probabilistic structural fire engineering, such as elements affecting post-flashover fire development factors (fuel load, ventilation, surface lining and compartment geometry), fire models, analysis methods and structural reliability. Epistemic uncertainty and aleatory uncertainty are investigated in the thesis by examining the uncertainty associated with modelling and the factors that influence post-flashover development of fires. A survey of 12 buildings in Christchurch in combination with recent surveys in New Zealand produced new statistical data on post-flashover development factors in office buildings in New Zealand. The effects of these parameters on temperature-time profiles are evaluated. The effects of epistemic uncertainty due to fire models in the estimation of structural response is also calculated. Parametric fires are found to have large uncertainty in the prediction of post-flashover fires, while the BFD curves have large uncertainties in prediction of structural response. These uncertainties need to be incorporated into failure probability calculations. Uncertainty in structural modelling shows that the choices that are made during modelling have a large influence on realistic predictions of structural response.

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NOMENCLATURE

Greek Symbols

λ_i	[W/mK]	The thermal conductivity of the layer i
λ_p	[W/mK]	The thermal conductivity of the fire protection material
ρ_a	[kg/m^3]	The unit mass of steel
ρ_i	[kg/m^3]	The density of the layer i
ρ_p	[kg/m^3]	The unit mass of the fire protection material
ρ_s	[kg/m^3]	The density of steel
σ	[W/m^{2}K^4]	Stephan Boltzmann constant (=5.67×10^{-8})
Φ	[-]	The configuration factor
Γ	[-]	Fictitious time factor
Δt	[second]	The time interval
$\theta_{a,t}$	[°C]	The steel temperature at time t
$\theta_{g,t}$	[°C]	The ambient gas temperature at time t
$\Delta\theta_{g,t}$	[°C]	The increase of the ambient gas temperature during the interval
ψ_i	[-]	Combustion factor

Alphabet Symbols

A_f	[m^2]	Floor area of a compartment
A_{fi}	[m^2]	Horizontal burning area of fuel
$A_{fi,max}$	[m^2]	Maximum horizontal burning area of fuel
A_j	[m^2]	The area of enclosure surface j, openings excluded
A_m	[m^2/m]	The exposed surface area of member per unit length
A_m/V	[m^{-1}]	The section factor for unprotected steel member
A_p	[m^2/m]	The appropriate are of fire protection material per unit length of the member

A_p/V	[m^{-1}]	The section factor for steel members insulated by fire protection materials
A_t	[m^2]	The total area of enclosure (walls, ceiling and openings)
A_v	[m^2]	The total area of vertical openings on all walls
b	[J/m^{2}s$^{1/2}$K]	$=\sqrt{\rho c\lambda}$, The thermal property of an enclosure
b_i	[J/m^{2}s$^{1/2}$K]	$=\sqrt{\rho_i c_i \lambda_i}$, The thermal property of enclosure surface i
b_j	[J/m^{2}s$^{1/2}$K]	The thermal property of enclosure surface j
C_a	[J/kgK]	The specific heat of steel
C_p	[J/kgK]	The specific heat of fire protection material
c_i	[J/kgK]	The specific heat of the layer i
d_p	[m]	The thickness of the fire protection material
h_{eq}	[m]	The weighted average of window heights on all walls
$h_{net,c}$	[W/m^2]	The net convective heat flux
$h_{net,r}$	[W/m^2]	The net radiative heat flux
h_{net}	[W/m^2]	The heat flux at the surface
M_c	[kNm]	The bending moment capacity of the beams at ambient temperature
$M_{k,i}$	[kg]	Mass of the material i
H_{ui}	[MJ/kg]	The net calorific values of the material i
O	[m$^{1/2}$]	The opening factor: $O = A_v.\sqrt{h_{eq}}/A_t$
Q	[W]	The rate of heat release of the fire
$q_{f,d}$	[MJ/m^2]	The design fire load density
$q_{f,k}$	[MJ/m^2]	The characteristic fire load density
$q_{t,d}$	[MJ/m^2]	The design value of the fire load density related to the total surface of the enclosure whereby $q_{t,d} = q_{f,d}.A_f/A_t$

LIST OF ABBREVIATIONS

AUC	-	Area Under Curve
CA	-	Cloud Analysis
CFD	-	Computational Fluid Dynamics
CIR	-	Cumulative Incident Radiation
DM	-	Damage Measure
DV	-	Decision Variable
EDP	-	Engineering Demand Parameter
FD	-	Fire Duration
FDS	-	Fire dynamic simulator
FORM	-	First–Order Reliability Method
FRR	-	Fire Resistance Rating
FSA	-	Fire Stripe Analysis
FSM	-	Fire Severity Measure
IDA	-	Incremental Dynamic Analysis
IFA	-	Incremental Fire Analysis
IM	-	Intensity Measure
LHS	-	Latin Hypercube Simulation
MARE	-	Mean Annual Rate of Exceedance
MC	-	Monte Carlo
MFT	-	Maximum Fire Temperature
MSA	-	Multi Stripe Analysis
MST	-	Maximum Steel Temperature
NZFS	-	New Zealand Fire Service
PBD	-	Performance Based Design
PBEE	-	Performance Based Earthquake Engineering
PSFE	-	Probabilistic Structural Fire Engineering

PEER	-	Pacific Earthquake Engineering Research
SORM	-	Second-Order Reliability Method
SSA	-	Single Stripe Analysis
TMFT	-	Time to Maximum Fire Temperature
TFM	-	Travelling Fires Methodology

1 INTRODUCTION

Fire can be devastating if not managed properly. A confined fire within a building is generally regarded as a compartment fire. Most structural damage occurs when a compartment is fully involved in fire (Castino and Harmathy, 1982). During this stage, the fire attains peak temperatures typically between 800°C and 1200°C. Conventionally, peak fire temperatures have been considered as the governing factors for structural fire design as mechanical strengths reduce with exposure to high temperatures. This has been the underlining principle for designs following the traditional structural fire design philosophy. The philosophy requires designers to follow a prescribed set of rules that have little or no scientific explanation but are practical ways that have been used to provide life safety. In prescriptive design, a building nominally rated to a given fire resistance rating (FRR) implies that all individual structural elements in the structure would survive exposure to the standard test fire for the specified duration of the FRR. This, however, is no indication of how each element, and therefore the overall structure would perform under real fire exposure. Prescriptive design is more about following code guidelines and keeping design recommendations within the specified limits (Buchanan, 1994; Buchanan and Abu, 2017). However, there has been a big paradigm shift in the fire engineering design space from prescriptive to performance based design. Performance Based Design (PBD) assesses how a building structure is likely to perform, given the potential hazard it is likely to experience. It allows the designer to formulate solutions to a problem which can be shown to meet performance goals as set out at the beginning of a project (Hadjisophocleous, 1998; Gibson, 1982). Performance based design focuses on the performance of a building which subsequently guides many design decisions that must be made. The process is iterative and starts with defining the performance objective. It is then followed by a preliminary design. Assessment of the design is performed in order to check whether it meets the performance objective or not. Finally, redesign and reassessment of the solution is performed until the desired performance level is achieved (Cornell and Krawinkler, 2000; Deierlein *et al.* 2003).

A performance based design framework has been adopted in a number of engineering fields such as earthquake engineering, fire engineering and wind engineering. Advancement in Performance Based Earthquake Engineering (PBEE) came from the Pacific Earthquake Engineering Research (PEER) center (Cornell and Krawinkler, 2000). The PEER PBEE approach has been adopted in structural fire engineering and taken many significant steps forward to develop a design process that relates a hazard to probable damage and losses. As the two hazards (fire and earthquake) are both low probability-high consequence in nature, the PBEE framework has so far been directly applied in structural fire engineering. However, research shows that there are problems with this direct application, as the quantification of the two hazards and their associated structural response are different (Moss *et al.* 2014; 2016). As part of this research, to promote the use of probabilistic methods in structural fire engineering, an investigation into the changes that are needed for the adoption of PBEE in structural fire engineering is also carried out.

There are various types of fires: fully developed compartment fires, travelling fires and localised fires which affect structural behaviour. For the response of structures to fires, fully developed post-flashover fires are normally used, as they tend to affect structures the most, due to their rapid rise in temperature over relatively short periods. Currently a structural fire design procedure with a performance-based approach comprises various types of design fires occurring at different locations of the building. In order to account for such probable events, multiple analyses (thermal and structural) are performed. The most appropriate way to account for these uncertain fires is probabilistic analysis. A probabilistic approach predicts a range of probable structural responses by accounting for the likelihood of different fire scenarios. A probabilistic design assessment of a building provides a measurable sense of reliability or risk of the structure for the multiple realistic fire scenarios which may occur in the structure during its lifetime. Recently, some progress has been made in the use of probabilistic methods for the analysis of structures in fire. Several probabilistic approaches are available. These include risk analysis, reliability analysis and Performance Based Structural Fire Engineering design, which is currently based on the PEER framework. This research focuses on the Performance Based Structural Fire Engineering design approach. For a more complete view of current probabilistic structural fire engineering the various probabilistic approaches are discussed in detail in Chapter 3.

The use of a probabilistic assessment in the performance-based design approach for fire engineering is termed Probabilistic Structural Fire Engineering (PSFE) and has been implemented by a number of researchers (Devaney, 2014; Hamilton, 2011; Lange *et al.* 2014; Moss *et al.* 2014; 2016; Rini and Lamont, 2008). PSFE investigates multiple fire scenarios which can occur in a structure and estimates probable structural response. Apart from the obvious advantages of the provision of life safety by assessment of structural resistance, PSFE also aids in financial loss estimation of buildings subjected to fires. As an adaptation of the earthquake engineering performance-based design framework, PSFE has provided the capacity for inclusion of the effects of uncertainties.

Probabilistic structural fire engineering begins with the quantification of a post-flashover fire in a simple but comprehensive manner. This simplicity allows distribution of fire scenarios to be generated based on post-flashover fire development factors such as fuel load, ventilation, room geometry and surface lining properties. Appropriate distributions of these factors are required to demonstrate current trends. These distributions are used to evaluate probable responses of the structure, from which the different fire scenarios that produce particular damage states may be quantified. This information is useful to designers as it allows quick decisions to be made on the expected performance of buildings under different fires and on whether a building should be repaired at moderate cost or replaced after a particular fire.

A key step in the probabilistic structural fire engineering process is to estimate the damage incurred to a structure. This is dependent on the structural response. In order to predict the structural response accurately, an estimation of the relationship between structural response and fire hazard is required. The establishment of this relationship is achieved by an analysis method. One such approach in PSFE is Incremental Fire Analysis [IFA] (Moss *et al.* 2014), where probabilistic distributions of structural response are computed at increasing levels of fire severity. In an earlier version of IFA by Moss *et al.* (2014), fire profiles were inappropriately scaled over a large range of fire severities and used in structural analyses. It was observed that a scaling adjustment was needed to ensure that scaled fires remain realistic for reasonable prediction of structural response. The scaling approach by Moss *et al.* (2014) implied that changing one characteristic of a fire profile (such as maximum temperature) affected other attributes of fire profiles such as time to reach maximum fire temperature, area under the curve and fire duration. Therefore, this research explores the direct implementation of the earthquake engineering analysis methods and also explores a new analysis method. The research draws comparisons to indicate which is more appropriate for structural fire engineering.

Another important step of the probabilistic approach is to recognize which fire hazard represents a fire scenario better. The identification of a suitable fire hazard indicator is guided by several criteria in PBEE such as efficiency, sufficiency, hazard computability and many more (Shome and Cornell, 1999; Luco and Cornell, 2007; Tothong and Luco, 2007; Giovenale *et al.* 2004; Kramer and Mitchell, 2006; Tothong and Cornell, 2007). The current research further explores these selection criteria and identifies a suitable fire hazard representor for structural fire engineering.

In earthquake engineering several experimental results are available (different magnitudes, ground shaking intensity, accelerations) which have produced earthquake hazard curves to evaluate the annual probability of exceedance of particular earthquake intensity, whereas no such historical records are available for fire due to limited experimental data. Therefore, it is difficult to calculate annual probability of exceedance data for any fire severity. Thus, current study demonstrates how the mean annual rate of exceedance of any particular value of FSM and corresponding structural response may be suitably generated for any location, using data collected by the New Zealand fire service for Christchurch city and New Zealand as a whole.

Structural response varies if the structure is modelled differently for analysis. The quantification of the difference in structural response due to different structural configuration is important because it provides information about the risk involved in the design solution due to the choice of the structural model. A noticeable variation in the probability of exceedance of structural response from different structural configuration may significantly affect the probability of failure which invariably governs the cost to reduce the failure risk. Therefore, this research compares the probability of exceedance of structural response given multiple fire scenarios derived using different structural models, 3D and 2D.

Uncertainty has a major influence on the assessment of reliability of a structure either aleatory (i.e. randomness in nature) or epistemic (lack of knowledge) uncertainty. The randomness in fuel load, ventilation, surface lining properties and compartment geometry are considered as aleatory uncertainty. It is observed that the mean value and standard deviation along with the distribution type of fuel load, room geometry, ventilation and surface lining properties available in literature are quite old and may not represent current situations. This has raised concerns about the suitability of the available distributions to new buildings. Therefore, there is a need to update the probabilistic distribution of post-flashover fire development factors through new surveys or combining available surveys. On the other hand, the epistemic uncertainty is due to the lack of knowledge. This uncertainty can be reduced by increasing the knowledge about the tools we use for structural and thermal analyses (i.e. fire models and structural models). The use of different fire models can produce different fire scenarios which can significantly affect the reliability of the structure (annual probability of structural response). Therefore, a careful selection of fire models is required to represent the probable fire scenario which could possibly occur in a structure. Therefore, there is a need to quantify the uncertainty fire models bring in during the calculation of structural response.

1.1 Research aims

The introduction of the PEER framework into PSFE has its own advantage over other probabilistic approaches. PSFE performs design of structures for life safety in addition to evaluating probable damage and losses. This study is part of a wide research vision which envisages accurate prediction of repair cost and repair time due to probable fire events. The research proposed here deals with the uncertainty involved in defining the probable fire scenario and the response of the structure. The research focuses on office buildings, for potential extension to other occupancies. This study is also a necessary step to help expand this field to evaluate the loss and damage states of structures for a given fire hazard. As there are discrepancies in the current application of the earthquake-engineering based approach, the thesis outlines key concepts of probabilistic structural fire engineering, and draws parallels with the more advanced Performance-based Earthquake Engineering (PBEE) methodology, to identify where improvements are needed. This sets the stage for the specific objectives of the research. These are:

1. Introduce and implement two new analysis methods in PSFE to relate fire hazard and structural response.

2. Investigate a set of potential fire hazard indicators and identify a suitable one amongst them which most suitably characterises fire severity.

3. Develop a methodology to predict the mean annual rate of exceedance of a given level of fire hazard and structural response for a major urban area.

4. Evaluate how uncertainty propagates through the use of various fire models and different structural modelling configurations on structural response.

5. Identify sources of uncertainty in different components of PSFE, e.g. post-flashover fire development factors, fire models, fire hazards, thermal analysis and structural analysis and propose solutions to help advance PSFE.

1.2 Outline of thesis chapters

This thesis is broken up into eight chapters as described overleaf. The material in the first six chapters of the thesis has also been published as three journal papers and three conference papers. The publications are listed at the beginning of each relevant chapter.

Chapter 2 - Literature review

A summary of structural fire engineering is presented. For a more comprehensive treatment of probabilistic structural fire engineering, the presentation includes fire models, post-flashover fire development factors, thermal analysis and structural analysis models. The current state of the art and the reasons why probabilistic structural fire engineering is needed are also discussed in brief.

Chapter 3 - Probabilistic structural fire engineering

Probabilistic structural fire engineering is discussed in detail in this chapter. Various aspects of performance based earthquake engineering such as analysis methods, intensity measures, engineering demand parameters and the adaptability of PBEE criteria into PSFE are all covered.

Chapter 4 - Fire Stripe Analysis And Clound Analysis

Analysis methods of PSFE are discussed in this chapter. Cloud Analysis and Fire Stripe Analysis are introduced in PSFE.

Chapter 5 - Efficient and Sufficient Fire Severity Measure and Annual Probability of structural response

The implementation of the PSFE approach is investigated by a case study of a steel-concrete composite beam exposed to a family of fires generated using currently available distributions of fuel load and ventilation. Various selection criteria of PBEE are implemented in PSFE to identify suitable fire severity measures (FSMs). To demonstrate the full application of the probabilistic framework the probability of occurrence of fires of given intensity in office buildings in Christchurch and New Zealand are calculated, based on existing fire statistics.

Chapter 6 - Effect of modelling on failure probabilities in structural fire design

Uncertainty in structural modelling is classified as epistemic uncertainty. The response of a structural element is different if modelled as an isolated member (2D) or as part of a structural system (3D). For probabilistic assessment of a given fire hazard, the variation in structural response due to different styles of modelling (2D or 3D) produces different annual probabilities of structural response. The quantification of this epistemic uncertainty is important because it provides information about the risk involved in the design solution due to structural modelling. This is demonstrated with the help of a case study of a composite steel beam, modelled both as an isolated element and as a part of a 3D structure, exposed to a suite of fire profiles. The annual probabilities of failure for both structural types are evaluated and compared to highlight the effects of structural modelling on structural response.

Chapter 7 – Uncertainty in structural response

The chapter presents new distributions for room geometry, surface lining properties and ventilation of office compartments, based on recent surveys. Fuel load distributions in various jurisdictions are also presented. Uncertainty in the use of different post-flashover fire models is investigated by the differences in structural response produced as a result of the different hazard models.

Chapter 8 - Conclusion

The conclusions from the various chapters are summarised in this chapter. Future work is also identified in the chapter.

2 LITERATURE REVIEW

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2.1 Post-flashover Fires

Fire is a process of combustion in which fuel is ignited and combined with oxygen, giving off light, heat and flame. In a compartment, a fire source transmits heat through radiation, convection and conduction, which significantly influences the thermal and mechanical behaviour of structural elements within that compartment and the overall building as a whole. Therefore, it is important to comprehend the growth of a fire inside a compartment. The complete fire development process is divided into different stages such as ignition, growth, flashover, fully developed, and decay as shown in Figure 2.1 (Buchanan and Abu, 2017). Fire begins with an ignition of an item. It then grows by spreading to adjacent combustible items. The growth process continues with the fire increasingly getting larger with the involvement of other items in the vicinity of the fire and is accompanied by the rapid production of hot smoke, which results in increasing temperature. In the transition of fire from the growth stage to the fully developed stage, the fire will normally continue to grow unless it is suppressed by fire extinguishment measures such as sprinklers or the action of firefighters. With the growth of fire, the temperature of the upper layer of the compartment increases, which subsequently increases the radiation to all available combustible objects in the compartment. The development of the fire from ignition until when the room is fully involved is known as the pre-flashover phase (Spearpoint, 2008). When the hot smoke layer reaches a temperature of approximately 600°C, or when the incident heat flux at floor level from the hot smoke layer reaches approximately 20 kW/m^2, flashover occurs, physically observed as the fully-involved phase. Beyond this the fire is said to be in the post-flashover phase, where peak temperatures develop in the room, followed by a decay phase. However, if the fuel is limited, or too remote from the original burning item, or if there is not enough air to support continued combustion, the fire may die before progressing to the post-flashover stage. The fully developed phase occurs after flashover and at this stage all fuel sources within the compartment take part in the fire and thus the fire may reach the maximum temperature of the compartment. The temperature in the compartment during this stage is often very high, i.e. in the range of 700°C to 1200°C.

At the fully developed stage the temperature in the compartment is controlled by the amount of ventilation available. A high amount of energy is released during this stage, which is limited by the supply of oxygen, and the rate of burning and heat release rate are characterized as ventilation-controlled (Karlsson and Quintiere, 1999).

Structural damage mostly occurs during the post-flashover stage. This is the case because structural elements lose significant amounts of strength when they heat up to temperatures above 300°C, which is more likely in post-flashover fires than in pre-flashover fires.

2.2 Post-flashover fire development factors

A severe fire may lead to the collapse of the structure. Various elements play an important role in the development of severe fires. These include ignition, flame spread properties of materials, the effect of active control measures, fuel load, ventilation, the occupancy type and size of the compartment (Buchanan and Abu, 2017). For post-flashover fires, these may be simplified into four key elements: fuel load, ventilation, compartment geometry and thermal properties of the enclosure boundaries - i.e. surface lining. Each is discussed in detail in the next sub-sections.

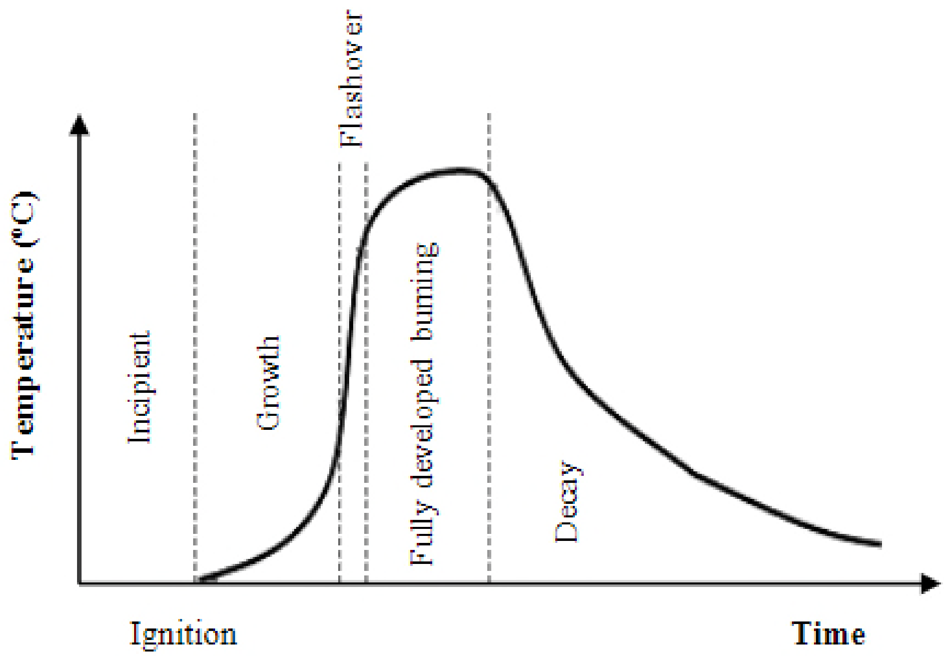

Figure 2.1: Fire growth in a compartment (Buchanan and Abu, 2017)

Fuel load

Amongst the above mentioned four post-flashover fire development factors, research recognizes fuel load as the factor which has the most impact on the behaviour of a structure under fire conditions (Ribeiro *et al.* 2016). Fuel load is the total heat content upon complete combustion of all the combustible material inside the fire compartment. For a typical compartment (see Figure 2.2) Eurocode 1 (CEN, 2002) defines the characteristic fuel load density per unit area as (Equation 2.1):

$$q_{fk} = \frac{1}{A_f}\sum(M_{k,i}.H_{u,i}.\psi_i) \tag{2.1}$$

Here $M_{k,i}$ is the mass of the material 'i' in kg, $H_{u,i}$ is the net calorific value of the material 'i' in MJ/kg, ψ_i is the factor describing the combustion behaviour of the material 'i' and A_f is the floor area of the compartment in m^2.

A review of surveys conducted on fuel load energy densities in office buildings over the past 50 years reveals that the average value ranges from 348 MJ/m^2 to 1298 MJ/m^2 (Caro and Milke, 1996; Culver and Kushner, 1975; Khorasani *et al.* 2014; Kumar and Rao, 1997; Narayanan, 1995; Thomas, 1986; Yii, 2000; Zalok and Eduful, 2013). Also, Eurocode 1 suggests a Gumbel distribution for fuel load in office buildings (see Figure 2.3) with an average value of 420 MJ/m^2 (CEN, 2002). This variation in average fuel load density indicates significant uncertainty which suggests its use as a random variable in design calculations as observed by a number of researchers (Devaney, 2014; Gernay *et al.* 2015; Guo and Jeffers, 2013; Iqbal and Harichandran, 2010; Khorasani *et al.* 2015; Moss *et al.* 2014; Ribeiro *et al.* 2016; Selamet and Akcan, 2015; Shi *et al.* 2013). For the same type and size of building fuel loads vary as a result of the specific use of each building, and there may be variations due to culture and geographical location. The variation in fuel load based on surveys at similar locations with similar occupancies but conducted at different times (years apart) indicates that fuel load values need to be updated from time to time in order to remain consistent with modern trends. The variation is too large to ignore. Therefore, a study which provides one distribution and mean value of fuel load for each major region and also provides one global distribution, applicable to any region, is helpful for precise probabilistic prediction of fire severity in compartments.

Ventilation conditions

Another factor governing compartment fire development is ventilation (see Figure 2.2). If the supply of air is limited then the growth of the fire is governed by the amount of oxygen entering the compartment for combustion. Although the maximum ventilation of the compartment has a fixed value due to construction (and can be easily calculated) the actual amount of ventilation during the fire is uncertain and it depends on the percentage of opened windows and glazing failure during the fire. Unlike fuel load, there is limited data available on ventilation area, which varies widely between different occupancies. There is limited literature for which ventilation conditions have been considered as a stochastic variable in probabilistic analysis (Devaney, 2014; Iqbal and Harichandran, 2010; Moss *et al.* 2014; Selamet and Akcan, 2015; Shi *et al.* 2013). According to the Eurocode parametric fire model the ventilation of a compartment is expressed as an opening factor, $O = A_v\sqrt{h_{eq}} / A_t$ in $m^{1/2}$, where A_v = ventilation area in m^2, h_{eq} = weighted average of window height in m and A_t = total internal surface area of the compartment in m^2 (CEN, 2002). The Joint Committee on Structural Safety [JCSS] code (Vrouwenvelder, 1997) suggests an equation (Equation 2.2) to account for the variation of amount of ventilation during a fire. This equation uses the maximum ventilation of the compartment and suggests a value of available ventilation (i.e. after breaking of window glass) during the fire event, which is a fraction of the total available ventilation.

$$F_v = F_{vmax}(1 - \xi) \tag{2.2}$$

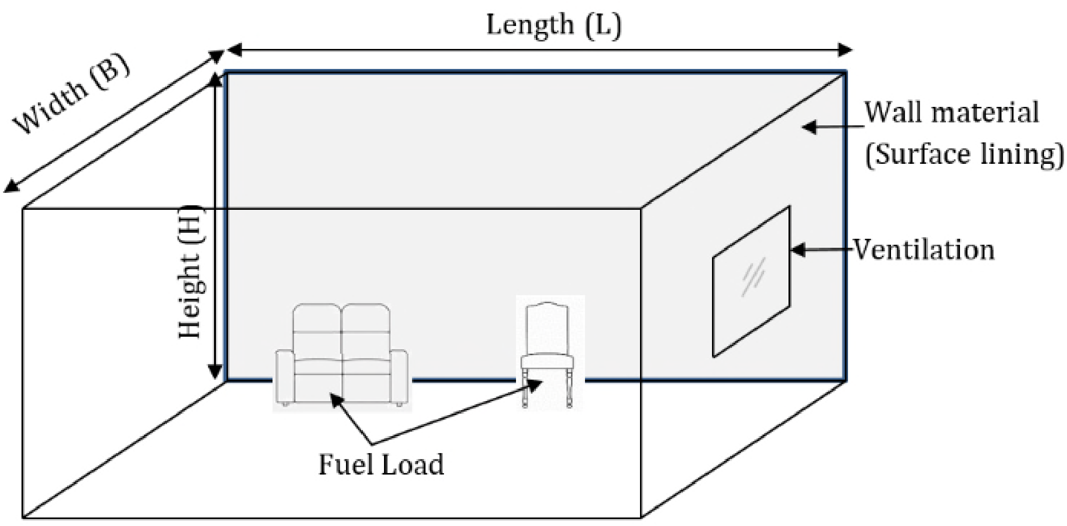

Figure 2.2: Fire compartment mentioning fuel load, ventilation, geometry and surface lining

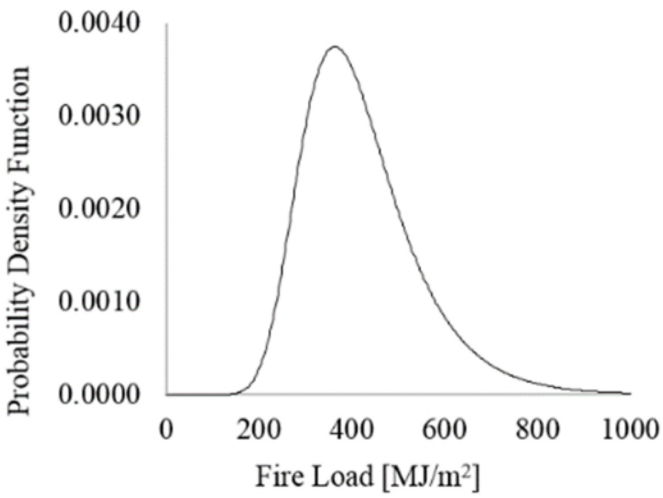

Figure 2.3: Probability density function of fuel load as per Eurocode 1 recommendation (CEN 2002)

Here, ξ is a reduction factor with a lognormal median of 0.2 and a dispersion of 0.2 and truncated at 1 to keep F_v positive which is the opening factor of the compartment while F_{vmax} is the maximum opening factor of the compartment.

Compartment geometry and Surface lining

In addition to fuel load and ventilation, room geometry and surface lining are also significant parameters of fire development (refer to Figure 2.2). Eurocode 1 suggests equations for calculation of thermal inertia (a measure of the heat stored by a material) of the compartment, which is made up of different layers of different materials lining each wall, floor or ceiling in the room (CEN, 2002). Architectural drawings of a building may help to estimate the values of the room geometry, ventilation, and surface lining. But this information is building specific, and each compartment has a different value of compartment geometry, ventilation and surface lining. This varies because no two compartments are the same. Most probabilistic studies on structural fire engineering have been building specific, and therefore, these key elements have been treated as deterministic (Gernay *et al.* 2015; Guo, 2015; Guo *et al.* 2013). Iqbal and Harichandran, (2010); Moss *et al.* (2014) and Selamet and Akcan, (2015) considered surface lining as uncertain in their studies, whereas only Selamet and Akcan, (2015) considered the variation in compartment geometry as random. All four factors (fuel load, ventilation, room geometry, and surface lining) vary significantly, as per the building usage, and also due to the location of the building. Therefore, there is a need to consider all of them as random variables in a single study to generate a realistic set of probable fire events which may occur in a structure during its lifetime but also provide some information for the investigation of the building stock in a community, city or country.

Once all the post-flashover fire development factors have been accounted for, fire models are used to produce temperature-time curves. The simplest way to express the fire scenario of the compartment through fire models in terms of temperature-time curve is by using one zone post-flashover fire models. Some examples of numerical formulations capable of generating post-flashover compartment fire models are Parametric fire (CEN, 2002), BFD curve (Barnett, 2002b), iBMB (Zehfuss and Hosser, 2007), Ozone (Schleich *et al.* 2002) and B-RISK (Baker *et al.* 2013; Wade *et al.* 2013). The process of generating a fire profile through each model, and associated assumptions and governing equations, are discussed below.

2.3 Post-Flashover Fire models

The objective of fire modelling is to mathematically represent as close as possible the effects of a real compartment fires. The more efficiently a fire and its effects can be modelled, the better the designer can predict the potential behaviour of the structure, and therefore minimize negative effects. Fire models generate the fire based on set assumptions and equations. Therefore, they are characterised based on their assumptions in the theory to evaluate the temperature of the compartment at any given time. There are several classifications of fire modelling such as nominal fire curves, parametric fire curves, zone modelling and field models.

Nominal fire curves are design temperature-time relationships based on furnace tests. Examples of nominal fire curves are standard fire, hydrocarbon fire and external fire (CEN, 2002). The standard ISO temperature-time curve (Equation 2.3) is widely used in structural fire design. In Equation 2.3, θ_g is the gas temperature in $°C$ and t is the time in minutes. The standard fire is very useful in fire-resistance tests but it does not represent real fires. In the Standard Fire curve, the furnace temperature rises continuously whereas in real conditions temperatures decrease after the consumption of fuel (refer to Figure 2.4).

On the basis of mass and energy balance equations, Magnusson and Thelandersson, (1970) published curves showing the development of real fires. These curves were adopted by Swedish standards and which served as the basis of Eurocode parametric fires (Devaney, 2014; Zehfuss & Hosser, 2007; CEN, 2002).

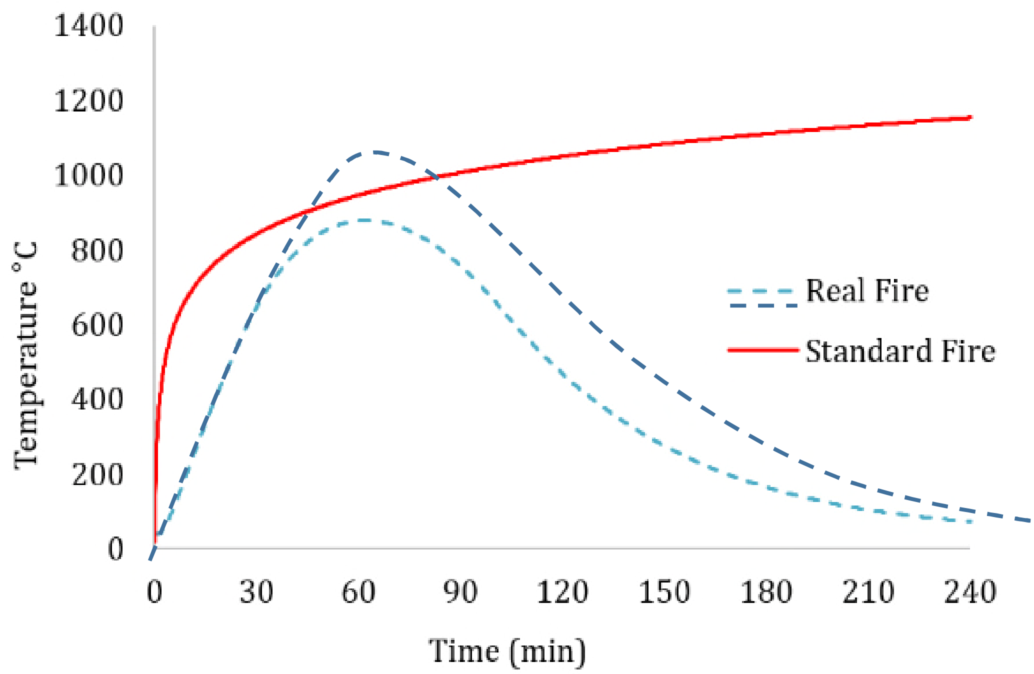

Figure 2.4: Standard fire and real fire curve

$$\theta_g = 20 + 345.\log_{10}(8t + 1) \tag{2.3}$$

Zone and Computational Fluid Dynamics (CFD) models are frequently used by designers for fire hazard predictions and fire safety designs. Zone models divide the fire compartment environment into homogenous layers, also recognised as control volumes. Zone models are categorised based on the temperature layer inside a compartment such as single zone, two zone and multi-zone models. In single zone models (as shown in Figure 2.5), the whole compartment is assumed to be of a uniform temperature layer such as the BFD Curve (Barnett, 2002b), COMPF2 (Babrauskas and Williamson, 1979) and the Eurocode parametric fire (CEN, 2002).

In a two-zone model (as shown in Figure 2.6), the upper layer comprises of hot smoke gasses released during a fire, and the lower layer contains colder ambient gasses; whereas a multi-zone model divides the compartment into several layers of temperature. Some zone models are CFAST/FAST (Peacock *et al.* 2013), OZONE (Schleich *et al.* 2002) and B-RISK (Baker *et al.* 2013; Wade *et al.* 2013).

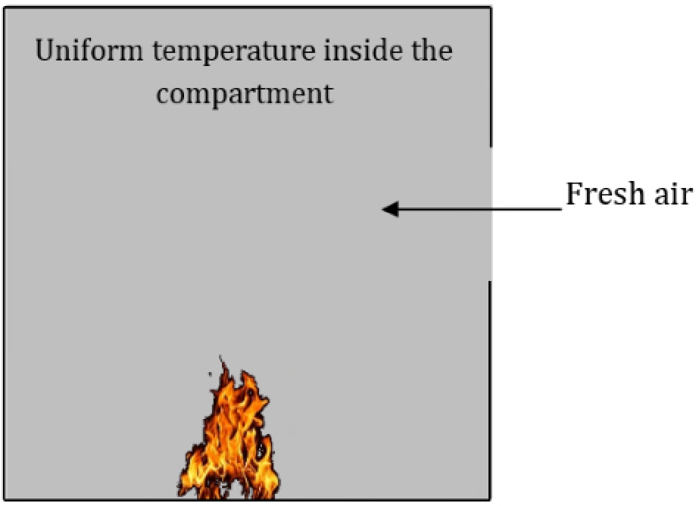

Figure 2.5: One Zone model

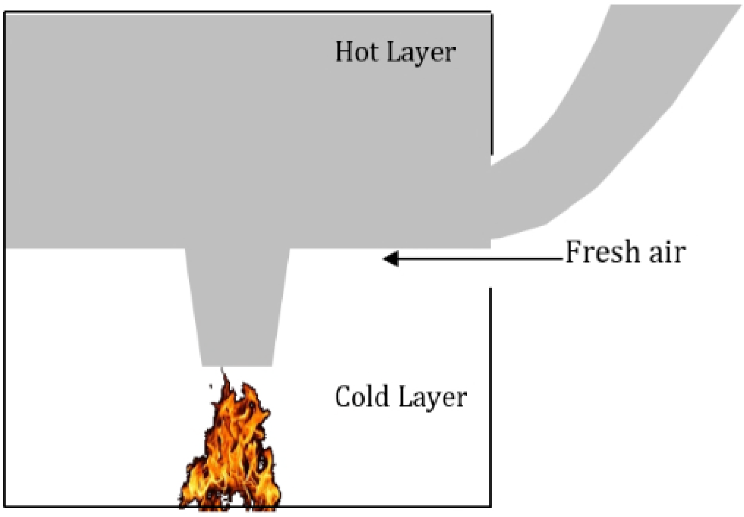

Figure 2.6: Two Zone model

In CFD models, such as the Fire dynamic simulator (FDS), three-dimensional geometry is used to define the fire environment (Bong, 2012; McGrattan *et al.* 2013). It is a field model which divides the compartment into a large number of control volumes and solves conservation equations (i.e. mass, energy and momentum) inside each control volume (McGrattan *et al.* 2013). Since FDS has more than two uniform zones, it is more suitable for complex geometries where two zones do not accurately describe the fire phenomenon. Though FDS models have the potential to examine the layer temperature at any point in the space, their complexity and computational time are considerably greater than other models. FDS is a very sophisticated modelling package and requires very detailed information of the fuel as an input such as radiative fraction and heat release rate of fuel material (Bong, 2012). The momentum conservation equations, along with mass and energy balance equations, are solved in CFD models during the analysis. One of the advantages of CFD models is that temperature can be estimated at any point in the space. However, the drawback in structural application is that it has high computational demands, which are more easily assessed if uniform temperature fires can be easily defined around the structural element. While zone modelling has many advantages, for example, flexibility and simplicity, there are various inherent limitations in the application of zone models, such as inappropriate representation of the fire scenario especially for large spaces (Klote and Milke, 1992).

B-RISK

B-RISK is a zone modelling software based on mass and energy conservation equations. In B-RISK, each compartment is divided into two homogeneous layers, a hot upper layer and relatively cooler lower layer. It deals in both pre-flashover and post-flashover conditions. B-RISK has the capability to switch from two zones to a one zone model based on the temperature and the radiant heat flux of the upper layer. It accounts for all the factors which influence the development of compartment fire such as fire load, ventilation, room geometry, and surface lining material (Wade *et al.* 2013).

B-RISK is a quantitative risk assessment tool, which incorporates probabilistic distributions of input parameters, deterministic calculations and Monte-Carlo iterative functionality. It deals with the risk and uncertainty inherent in performance based fire safety engineering (Baker *et al.* 2013). It includes an item-to-item fire spread module and a Design Fire Generator (DFG) allowing building contents to be randomly placed inside a room and the overall heat release rate to be determined (Frank, 2013). It produces outputs in the form of cumulative density functions of probability (Baker *et al.* 2013). The program allows results to be viewed in graphs or tabular form and will save results directly to an Excel spreadsheet with automatic generation of Excel charts for selected variables. B-RISK is different from other two-zone models because it allows users to assign statistical distributions to various fire safety system parameters, as well as the reliability and effectiveness of the systems. Wade (2013) compared and documented experimental data with the results of B-RISK and also showed the evaluation of the suitability of the model for the intended application.

Parametric Fire

Parametric fire takes into account important elements which influence the development of fire in the particular compartment such as fuel load, ventilation, compartment boundary material properties, and geometry of the compartment (CEN, 2002). They are based on the hypothesis that temperature is uniform in the compartment (i.e. single zone model), which limits their field of application to post-flashover fires in compartments. They have certain limitations to their use, such as applicability only for compartments of floor area up to 500 m^2, maximum compartment height of 4 m, no opening in the roof, thermal inertia ranges from 100 to 2000 $J/m^2s^{1/2}K$ and opening factor ranging from 0.02 to 0.2 $m^{1/2}$. However, PD-6688-1-2 (2007) allows the use of parametric fire curves up to 1000 m^2 based on comparisons of parametric fires with experimental data. Despite its limitations, it is a widely accepted one zone post-flashover model due to its suitability for stochastic analysis. It can also be modelled in a spreadsheet and permits Monte Carlo simulations to allow probabilistic analysis with variations in the post-flashover fire development factors as identified in Section 2.2. The generation of the temperature-time curve is performed in two stages: heating phase and cooling phase as documented in the Eurocode (CEN, 2002). To account for a compartment surface lining consisting of different layers of material, the Eurocode also suggests equations for calculating thermal inertia of the entire compartment based on a parameter 'b' whose value depends on the materials used to line the compartment.

BFD curve

The BFD curve has been developed from a series of large-scale tests performed at Cardington. The data produced from these tests were analysed and statistically investigated with curve fitting to produce the curve. The curve was later verified with other test results and it has successfully been fitted to 142 fire test results (Barnett, 2002a; Barnett, 2002b). The BFD curve model is a single log-normal equation used to represent decay and growth phase of a compartment fire. The main equation that describes the fire is as follows (Equation 2.4):

$$T = T_m e^{-z} + T_a \tag{2.4}$$

Here, T (°C) is the temperature at any time t (min), T_m is the maximum temperature of the compartment (°C), T_a is the ambient temperature (°C), and $z = (\log t - \log t_m)^2/s_c$, where s_c is the shape constant for the curve, and t_m is the time to reach maximum temperature (min).

To determine the maximum temperature of the fire, various post-flashover methods are available, but here the method of law has been proposed, due to its simplicity in application. It uses the fuel load mass density (ψ) and inverse opening factor ($\eta = 1/F_{o2}$) to calculate maximum temperature. Here F_{o2} is the opening factor.

Time to reach maximum temperature is based on the evaluation of fire type whether it is fuel controlled or ventilation controlled. Equations suggested for maximum temperature time requires the knowledge of total fuel load (E), fire intensity (Q), rate of burning growth (t_g) and decay coefficients (t_d). Another way, suggested by Barnet (2002a, 2002b) is to calculate the time to maximum temperature (t_m) from the Eurocode parametric equation (t_m = 60 t_{max}). Here t_{max} is calculated from the equation provided in Eurocode 1 Part 1.2 (CEN 2002). For shape constant s_c, Equation 2.5 is used for an uninsulated compartment.

$$s_c = 1/(4.F_{O2} + 0.1) \tag{2.5}$$

iBMB curve

iBMB curves are derived from the heat release rate of the design fire in contrast to the Eurocode parametric fire. iBMB connects the heat release rate with the temperature of the compartment. It divides the curve into three major components, which are based on the heat release rate (HRR) curve’s growth phase, constant phase and decay phase. The time up to the peak HRR is the first stage of the iBMB curve and it assumes quadratic growth in temperature during this stage. Secondly, the duration when the peak HRR is constant, i.e. up to the 70% of fuel consumption, the iBMB curve attains its peak value. The third stage is the decay phase of the curve. The governing equations for the generation of the temperature-time curve are shown in research presented by Zehfuss and Hosser (2007).

Accurate prediction of fire temperature profile and computational efficiency are important aspects of fire modelling. Considering the efficacy of the analysis, a simple representation of fire effects (fire curves, one zone models) seems reasonable for post-flashover investigation of structural behaviour. Therefore, various researchers in PSFE have used the Eurocode parametric fire as a post-flashover fire model (Hamilton, 2011; Lange *et al.* 2014; Moss *et al.* 2014). The comparison of fire curves from various fire models is available in the literature (Barnett, 2002a; Pope and Bailey, 2006). Fire models can significantly affect the resulting annual probability of fire hazard and then subsequently influence structural response.

Therefore, careful selection of a fire model is required to replicate the probable fire event that could occur in a structure.

2.4 Travelling fire

Traditionally, structural fire design is performed for the post-flashover fire which considers uniform temperature in the compartment and assumes that the fire is a worst-case scenario fire. In large compartments the burning of items in the whole compartment at once is very unlikely, therefore fire travels along the length of the compartment which results in a non-uniform temperature distribution within the compartment. A methodology called Travelling Fire Methodology (TFM) has been introduced by Stern-Gottfried and Rein (Stern-Gottfried and Rein, 2012a; 2012b) to accounts for this phenomenon in large compartments. The available travelling fire models in the literature are presented by Clifton in 1996 (Clifton, 1996) and later by Rein in 2012 (Stern-Gottfried and Rein, 2012a; 2012b). TFM is discussed in brief below.

In TFM, the compartment is divided into two zones as shown in Figure 2.7: far field and near field, and both regions constantly move along the length of the compartment. The near field is the temperature of the flames and the far field is the smoke temperature mixed with fresh cool air which reduces with distance from the fire. Both far field and near field temperature combine together to represent the progress of the fire temperature inside the large compartments (Dai *et al.* 2017).

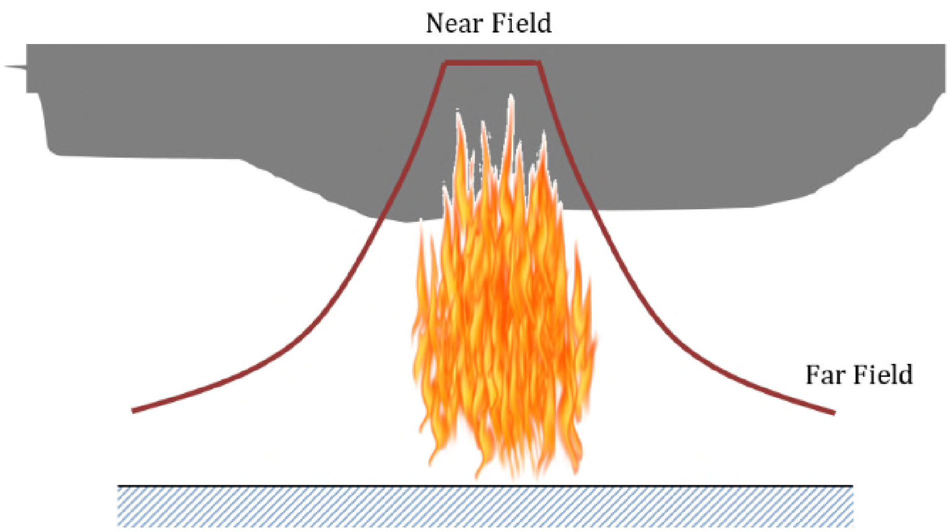

Figure 2.7: Travelling Fire

The focus of this research is to develop fire hazard representors, structural response parameters and analysis methods to help the probabilistic structural fire engineering process. Once the process is established, the information may be extended to travelling fires.

As discussed in Section 2.3, the parametric fire is a post-flashover fire model. PD-6688-1-2 (2007) permits the use of parametric fire for floor areas up to 1000 m^2. The current study focuses more on evaluating the critical response of structures when the compartment dimensions are less than 1000 m^2.

2.5 Thermal and structural analysis

In the presence of fire, the temperature of a structural element increases. This occurs because of the transfer of heat between the fire and the structural elements due to the difference in the temperature of fire and the structural element. Heat transfer occurs from high temperature area to low temperature area by various means such as conduction, convection, and radiation.

Conduction process occurs in solids and involves hot solid molecules passing heat energy to adjacent molecules. This development encourages the increase in temperature distribution within the solid materials. In structural fire design, when a structure is exposed to a fire, a rise in temperature of the member occurs. This happens due to the conduction of heat through the member as a result of the thermal environment. The thermal inertia of a material has a significant effect on the conduction process. For example, the lower the thermal inertia, the faster the heating. Convection occurs between fluids (i.e gases or liquids) and solids. In a compartment fire, the convention phenomenon involves heat transfer between the surfaces of the element and the surrounding gases. Radiation is the transfer of energy by means of electromagnetic waves through a vacuum. In a typical room fire, radiation is the dominant method for heat transfer from flames to fuel sources, from hot smoke gas to building boundaries, and from a burning building to an adjacent building.

The conduction process starts as a result of temperatures that have been induced at the surface of the structural member because of convective and radiative heat transfer from the fire itself to the member surface. As the resistance of the structural member is dependent on temperature, it is imperative to perform thermal analysis before structural analysis for steel and concrete elements. However, for timber structures fully coupled thermo-mechanical analysis is performed.

Thermal analysis is affected by the properties of material and the fire. Concrete and steel material properties vary with temperature as shown in the Eurocodes (CEN 2004, 2006). Specific heat and thermal conductivity are two thermal properties which significantly affect their heating rate in fire. The amount of heat required to heat a unit mass of the material by one degree is called the specific heat. Eurocode has suggested equations to provide the value of specific heat for each temperature increment. For steel, a sharp peak occurs around 735°C is due to a metallurgical change in the steel crystal structure. Thermal conductivity is a measure of the rate of heat conduction. The Eurocode suggests a linear approximation for most structural steel (CEN, 2004). Another thermal property is thermal elongation which is defined as the increase in length of the member divided by the initial length of the member. This specifies the free thermal expansion during heating.

Appropriate methods should be used for thermal analysis, such as the lumped mass capacity method for steel structures. This method calculates the average member temperature under quasi-steady conditions, ignoring a thermal gradient through the cross-section. In order to evaluate the member temperature both convection and radiation components of heat flux are calculated. The Eurocode (CEN, 2002) suggests an equation for calculating discrete increases in temperatures of steel, as a result of convection, radiation and conduction heat transfer.

Structural analysis of a thermally exposed member is performed to measure its load bearing capacity under elevated temperature situations. The loads, such as dead load and live load, are considered to be applied to the structure and the response of the member shows its stability under fire conditions. The response of the member is influenced by the thermal properties of the fire, thermal properties of the insulating material (if any), the mechanical properties of the member, and the design loads. These influential factors have high uncertainty and their effect on the response of the structure has been studied by many researchers (Gernay *et al.* 2015; Gernay *et al.* 2016; Iqbal and Harichandran, 2010; Kodur *et al.* 2010; Luecke *et al.* 2005; Olsson *et al.* 2017; Quiel and Garlock, 2010; Ribeiro *et al.* 2016). The uncertainty of these factors is expressed in terms of probabilistic models to measure the failure probability of a structure (Khorasani *et al.* 2015). Apart from the aforementioned sources of uncertainties, there are more segments in the design process, such as structural modelling and fire location in the structure, whose uncertain behaviour may affect the structural response. The inclusion of all sources of uncertainties enables the designer to quantify the likely variation in the structural response predicted from the analyses. This facilitates the evaluation of the level of risk involved in the design solution. There have been significant attempts made so far to cover a wide range of uncertainties. However, a comprehensive study that produces a structural response considering uncertainties from all sources together is still missing.

The understanding of the behaviour of a structure under fire conditions has increased significantly over the last two decades. The advancement in the use of finite element models facilitate the knowledge enhancement in the field of structural fire engineering. Analytical models are continuously growing in their computation power and becoming more robust. The most common finite element softwares for this purpose are SAFIR (Franssen, 2005; 2012), VULCAN (Bailey, 1995; Huang *et al.* 2003a; 2003b; 2004a), ABAQUS (ABAQUS, 2014), OPENSEES (McKenna, 1997, Jiang *et al.,* 2015) and ANSYS (ANSYS, 2012).

SAFIR and VULCAN both are non-linear finite element programs developed especially for thermal and structural analysis of composite structures in fire conditions. VULCAN has been developed at the University of Sheffield, UK. SAFIR has been developed at the University of Leige, Belgium (Franssen, 2005; 2012). Their shell elements are capable of modelling large-deflection behaviour. Both software consider non-linear temperature dependent material properties, thermal expansion and contraction of materials at elevated temperatures, stress-strain degradation, cracking and crushing of concrete slabs, and yielding of reinforcements. This makes them capable of modelling composite steel-concrete floors at elevated temperature.

OpenSees is an open-source object-oriented software developed at the University of California–Berkeley (McKenna, 1997). OpenSees is an advanced finite-element computational tool for analysing the nonlinear response of structural and geotechnical systems subjected to seismic excitations. The University of Edinburgh's research team worked on adding structural fire modeling capability in Open-Sees (Jiang *et al.,* 2015). This extension to OpenSees was added to enable 2D thermomechanical analysis which involved creating a new thermal load pattern and temperature dependent properties.

2.6 Behaviour of Composite structures in Fire:

Various experiments and real fire incidents have enabled better understanding of structural fire behaviour of steel, concrete and composite construction. The Broadgate fire incident which occurred in London in 1990 (SCI, 1990) highlighted the robust response of composite structures despite many structural elements reaching their limiting temperatures during the incident. This inspired a series of large-scale experiments on an 8-storey steel building in the Cardington laboratory of the Building Research Establishment (BRE) in England from 1995 to

2006. Several studies have since been conducted to replicate the structural fire scenario of the Cardington tests and to understand structural behaviour under fire conditions (Wang *et al.* 1995; Bailey & Moore 1999; Sanad *et al.* 2000; Elghazouli & Izzuddin 2001, Gillie *et al*. 2001, Gillie *et al*. 2002, Usmani *et al*. 2000). The building was of composite construction and comprised of steel beams and columns and reinforced concrete slab. The beams were generally unprotected while columns were protected. Several tests were conducted with fire temperatures reaching beyond 1000°C, causing very high deflection but no structural collapse.

The behaviour of structural elements in fire is far more complex when they are part of a structure instead of being isolated. It has been studied by number of researchers (Usmani *et al.* 2001, Lamont *et al*. 2004, Usmani & Lamont, 2004). When composite structures are exposed to fires, at the initial stage restrained axial compression is induced in the steel beam due to its thermal expansion which causes lower flange of the steel beam to buckle. A similar behaviour was observed in the Cardington test as well (Wald *et al.* 2006). Later, with the increase in the temperature catenary action prevails in the steel beam and restrained axial compression declines. As deflections increase, and moment capacity decreases with the gradual weakening of the beam, the beam eventually carries load primarily due to catenary action. Tensile forces increase during the cooling phase when beams contract against the supports. This may lead to connection failure, which was evident in the Cardington experiments.

On the other hand, composite slabs perform well in fire under large deflections, they are able to carry large loads through tensile membrane action, which occurs due to the development of in-plane forces within the depth of the slab. The composite slabs experience tension in the center and a compression ring at the perimeter to support large vertical displacements. During tensile membrane action of the slab the reinforcement utilises its full tensile capacity to support the load.

In recent decades the understanding of the fundamentals of structural behaviour under fire has greatly increased due to the large amount of research carried out in the field as discussed above.

2.7 Paradigm shift from Prescriptive to Performance based design

A paradigm shift in structural fire design from prescriptive design to performance-based design not only allows the designer to think beyond code provisions to assess compliance, but also produces structures with quantifiable levels of safety that meet other design objectives beyond life safety. This paradigm shift results in the provision of only the required elements of construction. In some cases, this results in reducing the amount of required fire protection, while keeping or improving the desired performance level. Prescriptive design requires strict adherence to a set of rules that have little or no scientific justification but are practical ways that have traditionally been used to provide safety. Performance-based design, on the other hand, requires the designer to demonstrate that required performance criteria are met by the selection of relevant fire scenarios, acceptance criteria, and processes that adequately model the thermo-mechanical behaviour of the structure. This process is now more widely known as structural fire engineering, as the structure can be "engineered" to behave in a particular manner once its behaviour can be reasonably predicted. Both approaches tend to meet the primary objective of providing life safety. Following the prescriptive approach, a building nominally rated to a given fire resistance rating (FRR) will ensure that all individual structural elements in the building are rated to that FRR. In contrast, the performance-based approach will investigate a number of scenarios which challenge the design of the building, but must provide reasonable levels of safety. Even though the benefits of performance-based design are obvious, the prescriptive approach is frequently used in practice, because it is simple to implement. The initial design cost of the prescriptive designs is less compared to performance-based design and contains less complex design processes. However, there could be issues with performance based design in defining performance objectives since it involves various stakeholders and requires an agreed way forward with their acceptance.

2.8 Necessity of Probabilistic Structural fire engineering

Current structural fire design practice with a performance based approach involves multiple fires in different locations of the building, or different fires at the same location. This is typically achieved by multiple analyses of fires, their thermal insults on the structure and the subsequent mechanical response of the structure. As none of these fires are guaranteed to occur in a building, it is prudent to develop probabilistic methodologies to assess structural behaviour in fire conditions (Figure 2.8). A probabilistic approach accounts for the likelihood of different fire scenarios in addition to the full range of predictions of structural response (temperature,

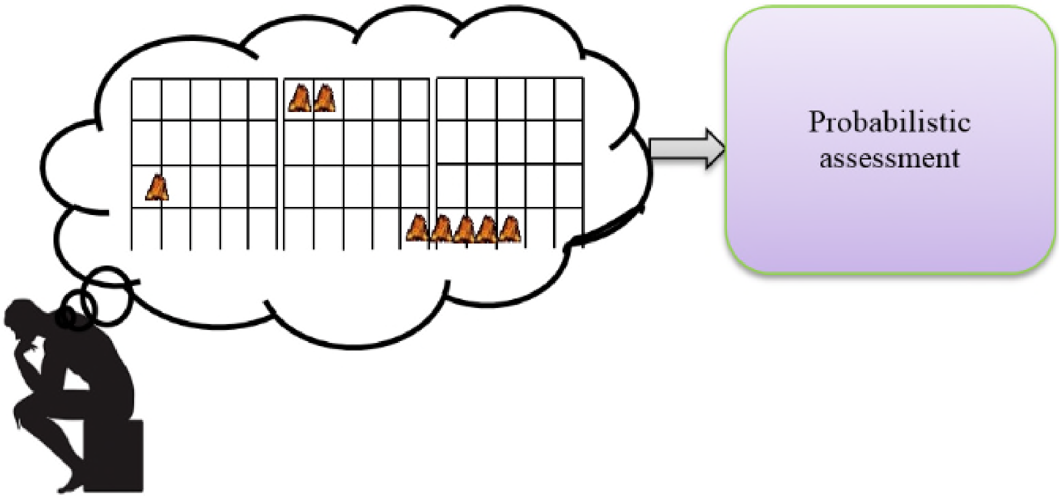

Figure 2.8: Uncertainty in fire occurence

deflection, failure time, etc.). Besides life safety, there are other design objectives that are not specifically accounted for in design codes around the world, but maybe important to various stakeholders, such as building owners, insurers, tenants, and the economy. These include loss of function of the building, reparability and other economic considerations. As a by-product of using probabilistic approaches damage analyses and financial loss assessments of buildings can also be carried out, thereby helping to meet the other important objectives beyond life safety.

Probabilistic performance based design of buildings is an effective and more comprehensive approach for assessing the performance objectives of structural systems. The probabilistic performance based design framework has been implemented in various engineering fields such as earthquake (Cornell and Krawinkler, 2000; Deierlein *et al.* 2003), blast (Hamburger and Whittaker, 2003), tsunamis (Riggs *et al.* 2008), hurricanes (Barbato *et al.* 2011; Barbato *et al.* 2013) and wind (Petrini and Ciampoli, 2012). It is progressively being adopted in structural fire engineering (Devaney, 2014; Hamilton, 2011; Lange *et al.* 2014; Rini and Lamont, 2008).

3 PROBABILISTIC STRUCTURAL FIRE ENGINEERING

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Many researchers have contributed towards the development of the probabilistic approach in the structural fire engineering field. Various aspects of probabilistic analysis have been explored to produce more reliable, accurate and useful results. The different approaches in the probabilistic domain are risk analysis, reliability analysis and performance based probabilistic structural fire analysis.

Risk is a subjective matter and requires agreement between different stakeholders to set acceptable risk standards or follow socially acceptable risk. De Sanctis *et al.* (2011) recommend a risk-based methodology based on a Bayesian probability network. A new risk based framework has been developed by Kirby *et al.* (2004) to specify fire resistance which is linked to building height and occupancy type. Another approach in the risk analysis domain is Probabilistic Risk Analysis (PRA) which has gone quite far in its development and has been adopted in BSI (PD 7974-7, 2003). PRA evaluates the risk of the design solution by calculating consequences and associated probabilities. The acceptance of a design solution is based on the comparison of the risk with acceptance criteria. Acceptance criteria can be "a low as reasonably practicable" (ALARP) which may not require an implicit evaluation of a cost-benefit analysis (CBA) implicitly. Therefore, safety targets are required to be considered during the analysis (Hopkin *et al.* 2017). The JCSS code has defined target safety levels which consider CBA and ensures adequate safety (Vrouwenvelder, 1997).

On the other hand, structural fire problems may be analysed by considering the level of safety required (or reliability), which is the core of reliability analysis. Reliability of a structure can be defined as the probability to demonstrate the achievement of the intended behaviour of a structure under a given condition. A structure may be vulnerable to a certain severity of fire. However, the same severity may not produce similar damage to other structures. This indicates that the severity of a fire is structure dependent and it is more convenient to set safety reliability targets for all structures to achieve during extreme loading conditions. The presence of uncertainty gives birth to the question of reliability. How reliable can the structure be if the calculation of the structure's capacity and demand are not certain? Therefore, as discussed in Section 2.8 an appropriate way to explicitly consider this uncertainty is to calculate capacity and demand in terms of probability considering them as random variables. In ambient design conditions, the application of the reliability targets demonstrates that safety targets are met considering future costs and benefits of safety investments. Van Coile *et al.* (2017) demonstrated that the implementation of ambient reliability targets to fires have issues and require more refinement. The ambient reliability targets cannot be processed to use as a function of fire occurrence rates in fire engineering scenarios. However, the concept of cost and benefits of safety investment might be applicable conceptually.

A pertinent methodology of measuring the reliability of a structure is to estimate the probability of failure. Probability of failure is not only a reliable indicator but also a valuable tool from a design point of view. The probability of failure can be calculated with the help of a First–order reliability method (FORM), second-order reliability method (SORM) or Monte Carlo approach. FORM underestimates the failure probability results but the observed error is not significant. The reliability of structural members using the Latin Hypercube simulation (LHS) and the first/second-order reliability methods (FORM/SORM) has been investigated by Guo *et al.* (2013) and Guo and Jeffers (2014), respectively. The failure probability can be improved by second-order approximation with SORM. On the other hand, Monte Carlo approaches are time-consuming but may incorporate uncertainty at various levels of the framework (Balogh and Vigh, 2016; Guo and Jeffers, 2013; 2014). Shi *et al.* (2013) and Guo *et al.* (2013). Van Coile *et al.* (2014) compared structural fire safety with structural design alternatives based on reliability evaluation. For reinforced concrete and prestressed concrete beams, reliability analyses were performed by Eamon and Jensen, (2013). A new and unique approach is proposed to deal with the member temperature estimation as well as recommend a protection thickness for all possible scenarios to prevent structural members exceeding their critical temperatures within the required period (Kirby *et al.* 2004 and Law *et al.* 2015). The method calculates fire resistance period based on time equivalence, which is known to have its flaws. Many time equivalence approaches use empirical correlations which account for fuel load, ventilation conditions, compartment size, lining materials and structural materials. Whilst they ease the determination of FRR, many parameters also affecting the structural failure are not explicitly considered. These include load ratio, member size and reinforcement size in reinforced concrete (RC) members. A change in any of these will alter the structural resistance of a member. However, it is not reflected in the existing empirical correlations. The Monte Carlo approach is used for probabilistic investigation and risk/failure/reliability is calculated.

Iqbal and Harichandran (2010) developed probability-based load and resistance factors for structural fire design. Lange *et al*. (2014), Moss *et al.* (2014; 2016) and Hamilton (2011) executed a performance-based design approach for structures in fire which is based on the performance based earthquake engineering framework developed by the Pacific Earthquake Engineering Research (PEER) Center.

These different probabilistic methods have developed substantially in recent times. All the methods have ways to incorporate various design objectives and produce solutions to meet those objectives. Each method has its own advantages and disadvantages. Risk analysis informs the designer about the associated risk of each design alternative and requires a particular design solution to be selected. The classification of risk is done differently by different people, which is the disadvantage of this process (Devaney, 2014), whereas Reliability methods have predefined safety targets which design alternatives have to meet. These safety targets contain some uncertainty in them and may compromise the accuracy of the results. Another probabilistic approach is the performance-based design approach, which not only designs structures for life safety within a linear framework but also aids in evaluating probable damage and losses. The research presented here follows the probabilistic performance based design approach. The following sections will provide a review of the PEER PBEE approach and how it is applied in structural fire engineering.

3.1 Performance based Earthquake Engineering (PEER framework)

The Pacific Earthquake Engineering Research (PEER) centre has developed a Performance-based Earthquake Engineering (PBEE) approach for seismic performance assessment of structures. The approach is characterized by four stages: hazard analysis, structural analysis, damage analysis and loss analysis, as shown in Figure 3.1. These assessments are linked by pinch variables that assess the likelihood of an event based on the occurrence of another. In the PEER framework, they are identified by the annual probabilities of exceedance of an event. The variables associated with each of the four stages are: Intensity Measure (IM), Engineering Demand Parameter (EDP), Damage Measure (DM) and Decision Variable (DV) (Cornell and Krawinkler, 2000).

In PBEE, the Intensity Measure (IM) is selected to efficiently and sufficiently describe the severity of an earthquake. Some examples of IMs are spectral acceleration, peak ground acceleration and peak ground velocity. Engineering Demand Parameters (EDPs) are variables which quantify the key response of the structure, such as inter-storey drift ratios, inelastic deformations, strains and floor acceleration. Damage Measure (DM) is a variable which provides information about the level of damage to the structure as a result of the building's response, and can generally be used to identify the type of repairs required and the functionality of the structure. Finally, Decision Variables (DVs) are described based on the performance measure of greatest interest to the stakeholders, such as cost or downtime (Moehle and Deierlein, 2004; Porter, 2003).

The PEER center proposed framework in Figure 3.1 can be expressed on the basis of a total probability theorem as stated in Equation 3.1:

$$\lambda(DV) = \iiint G(DV|\mathrm{DM})|\mathrm{dG(DM|EDP)}|\mathrm{dG(EDP|IM)}|\mathrm{d\lambda(IM)} \tag{3.1}$$

where dλ(IM) relates the ground motion IM to its mean annual frequency (MAF) of exceedance. dG(EDP|IM) and dG(DM|EDP) are the derivatives of conditional probabilities relating one quantity to another. dG(EDP|IM) is the derivative of the probability of exceeding a specified EDP value subjected to a specified IM value. Similarly, dG(DM|EDP) is the derivative of the probability of exceeding a specified DM value subjected to a specified EDP value. G(DV|DM) is the conditional probability that DV exceeds a given value with respect to DM value. The quantity on the left side of Equation 3.1, λ(DV), is a probabilistic description of the DV, such as the mean annual frequency of the DV exceeding a specified value. Equation 3.1 comprises pair-wise sequences of the four variables IM-EDP-DM-DV. The key assumption of the equation is that the last component is sufficient to describe the current step (Cornell, 2004). For example, in the damage analysis stage, EDP is sufficient enough to characterize DM and thus the IM does not need to be considered, which in mathematical terms corresponds to P(DM|EDP,IM) = P(DM|EDP) and DM is thus a function of EDP only.

In its standard form (Equation 3.1), the process computes annual probabilities, but the process can be easily modified to compute the probability of exceedance of different hazards during a certain period most notably the design life of the structure. Equation 3.1 has since been extended to include an additional integral to derive the mean value of an annual decision variable in the form of Expected Annual Loss (EAL).

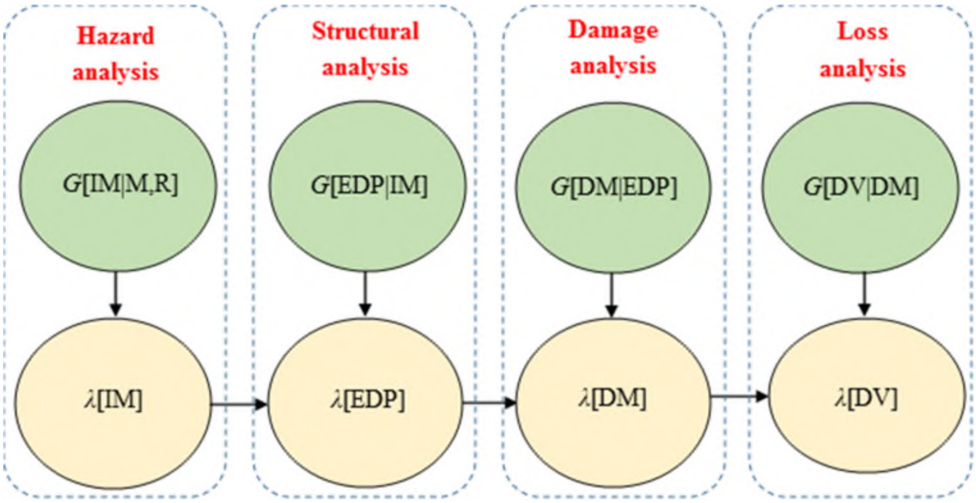


Figure 3.1: PBEE framework (Moehle and Deierlein, 2004)

This is a measure that can be expressed in terms of dollars, and offers a rational way to communicate the seismic vulnerability of a structure to its non-technical stakeholders (Dhakal and Mander, 2006).

3.2 Intensity Measure (IM)

The IM is a scalar or vector parameter which expresses the event intensity and which quantifies the rate/probability of exceeding a given intensity per year. It captures the attributes of ground motions which consequently affect the performance of the structure. A suitable IM may be defined based on ground motion parameters such as amplitude, acceleration, frequency, velocity and duration that significantly affect the response of the structure. A few examples of IM in PBEE are spectral acceleration, peak ground acceleration and peak ground velocity. Some of the IM values are of great importance to a designer, such as how a structure will perform at a spectra acceleration at natural frequency of structures $Sa(T_1)$. Therefore, these significant IM values are set as a target IM value.

The selection of an optimum IM, which describes the severity of an event effectively, is based on certain properties. Presently, IMs need to meet the following criteria (Giovenale *et al.* 2004; Tothong and Cornell, 2007):

- Efficiency
- Sufficiency
- Scaling robustness

- Hazard Computability
- Predictability

Efficiency gives a measure of correlation of IM with EDP. An efficient IM provides relatively smaller dispersion of the response of the structure. It is a measure of variance in the EDP-IM relationship. The smaller the dispersion, the better the efficiency of the IM. A better IM therefore reduces the number of analyses required to achieve a desired level of confidence in the response of the EDP (Mackie and Stojadinović, 2003).

An IM is called sufficient when the structural response (i.e. EDP) given a particular IM are independent of magnitude (M) and source distance (R) of the earthquake (Padgett *et al.* 2008). The residual is the error in the prediction by the regression model with respect to the computed EDP, i.e. predicted value minus the computed value. No trend in the residual plot as a function of ground motion parameters (M and R) indicates the sufficiency of an IM.

Another property of a good IM is that when records are scaled to a target IM value, the IM value results in an unbiased structural response compared to unscaled ground motions (Tothong and Luco, 2007). Estimation of scaling robustness for a given IM level can be done by conducting regression analysis on the scaling factor versus structural response. In general, a smaller slope of the regression line indicates more robust scaling, and a larger slope indicates less robust scaling (Lin, 2008).

The choice of an IM is also based on hazard computability. This is defined as the level of effort required to determine the severity of the hazard in terms of the proposed IM. Hazard curves are readily available in terms of peak ground acceleration or spectral acceleration but some IMs require considerable efforts for their determination (Giovenale *et al.* 2004).

Predictability (Kramer and Mitchell, 2006) is the accuracy with which the IM can be calculated. If the predictability of an IM is poor, then the accuracy in predicting the seismic response (for a given earthquake scenario) will also be poor. This IM property has been implemented in selecting the appropriate IM for seismic hazard scenarios (Bradley *et al.* 2009a; Bradley *et al.* 2009b).

Apart from the abovementioned selection criteria there are other selection criteria such as practicality (Padgett *et al.* 2008), effectiveness (Mackie and Stojadinović, 2003) and proficiency (Padgett *et al.* 2008), which are also important for the identification of the suitable

IM, but these are not specifically addressed in this thesis. Th use of hazard computability criteria is subjective. An FSM which is difficult to compute manually can be computed quickly with the use of software application. From PBEE studies, it was observed that efficiency and sufficiency are the two important selection criteria to suitably represent a hazard scenario. The other selection criteria are a fairly new development in PBEE. The implementation of the selection criteria in PSFE is discussed later in Section 3.9.

3.3 Engineering Demand Parameter

Engineering Demand Parameters (EDPs), are parameters that describe the response of the structure and can be used to estimate damage caused to the building's components. An ideal EDP correlates best with damage to the structure at the required performance level (Bachman, 2004). EDPs are generally classified into two groups, local and global (Freddi, 2012; De Biasio, 2014). Local EDPs are defined at a component level. Examples are deflections, strains and stresses. Global EDPs, on the other hand, take whole structural response into account; e.g. peak inter-storey drift, inelastic component deformations, and floor acceleration. Presently, inter-storey drift is the most commonly used EDP.

3.4 Damage Measures and Decision Variables

Damage Measure (DM) classifies the level of damage to the structure such as no damage, replacement of structure and collapse which could be either local or global. Once the damage is identified and the repair strategy is selected, the Decision Variable estimates the cost to repair with each associated DM in terms of downtime or dollar estimates the annual impact of the earthquake to the structure

A key step in the PBEE process is to estimate the damage incurred to a structure. This is dependent on the structural response (EDP). In order to predict the structural response accurately, an estimation of the relationship between structural response and hazard is required. The establishment of this relationship is achieved by an analysis method. Various analysis methods are available in PBEE as shown in Figure 3.2.

3.5 Analysis methods

With the analysis methods, structural response is estimated for the given hazard and the results are processed to predict the probabilistic response of a structure. To obtain this prediction and establish a desired relationship between EDP and IM, the analysis methods available, as shown in Figure 3.2, are Single Stripe Analysis [SSA] (Jalayer, 2003), Multi Stripe Analysis [MSA]

(Bazzurro *et al.* 1998; Jalayer and Cornell, 2009; Baker, 2007), Incremental Dynamic Analysis [IDA] (Vamvatsikos and Cornell, 2002) and Cloud Analysis [CA] (Bazzurro *et al.* 1998; Jalayer, 2003; Luco and Cornell, 2007). These methods are fairly well developed and are in use based on requirements of accuracy and computational effort.

Single Stripe Analysis [SSA]

In SSA, ground motion records are scaled to a target IM value and structural response (EDP) is recorded for non-linear analyses of scaled records of the intensity measure. Median and standard deviation (dispersion) of the EDP for the given IM is calculated. Since records are scaled to only one level, there is no slope information between EDP and IM, therefore sensitivity of the structural response to a change in intensity measure, is assumed to be unity i.e. b, as a coefficient of local slope. The single stripe method (Jayaler, 2003) requires less computation, but it provides no information of “*b*”, which implies that the median structural response and intensity measure have a relationship in arithmetic space.

Multiple Stripe Analysis [MSA]

Multiple stripe analysis (Bazzurro *et al.* 1998; Jayaler and Cornell, 2009; Baker, 2007) is a collection of single stripe analyses performed at various levels of intensity measures. IM at different levels can be achieved by scaling ground motion records or by selecting different records with particular IM levels. Structural response (EDP) is calculated for the IM at all levels. This method uses both scaled and unscaled ground motion records. Each IM level is analysed as a single stripe and the mean and standard deviation of each stripe is computed. Different ground motions may be used at each IM level. Therefore, prediction of increasing fractions of collapse with increasing IM is not possible. Since it involves scaling and analysis for each IM value, the computational time increases.

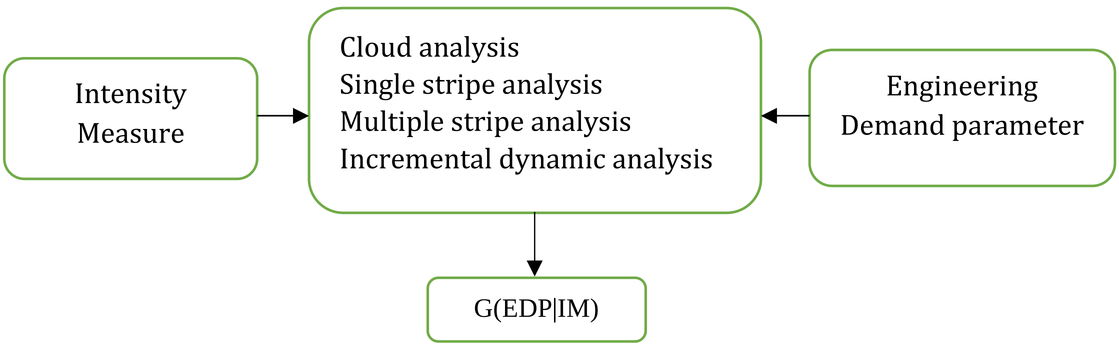


Figure 3.2: IM-EDP analysis methods

Incremental Dynamic Analysis [IDA]

IDA (Vamvatsikos and Cornell, 2002) is an approach in which nonlinear dynamic analysis is performed on a structure under a suite of ground motion records. Each record is scaled to several IM levels, producing structural responses for each IM. These responses are connected with a line or curve (mainly spline) at every level and IDA curves are generated for each record. Scaling of records is performed in order to capture the response of the structure from elastic to a certain limit state capacity. IDA has the capability to predict the collapse of the structure for a particular earthquake. Continuity in the IDA curve indicates the use of the same records at all IM levels, rather than selecting different earthquake records to incorporate different casual earthquake events.

IDA and MSA are called wide range methods and are very computational intensive. Cloud analysis and SSA are called narrow range methods and are cost effective. IDA and MSA both require scaling of records and scaling can be avoided in MSA by careful selection of records which provides IM at all levels. Selection of ground motion intensity measure (IM) is an important step in the probabilistic evaluation of the seismic response of the structure.

Cloud Analysis [CA]

Cloud analysis involves the non-linear analysis of the structure subjected to a set of unscaled ground motion records. The response of the structure (obtained from the analysis) is plotted against intensity measure. The wide scatter of information (each response corresponding to a given IM) presents the results in a form of a "cloud" – hence the name. Linear regression analysis in the natural logarithmic scale is performed in order to estimate the mean value and standard deviation of the EDP given the IM value. The cloud method (Bazzurro *et al.* 1998, Jayaler, 2003, Luco and Cornell, 2007) has various merits and limitations. These include:

- It presumes a linear relationship between EDP and IM with constant variance.
- It is appropriate over a limited range of IM levels.
- As this method uses the 'as provided' ground motion records it has the benefit of reducing computational time.

Currently, PBEE is well developed and widely adopted. Both earthquake and fire are low-probability and high-consequence hazards. Research has shown that the Performance-Based Earthquake Engineering (PBEE) framework can be adopted in structural fire engineering

(Hamilton, 2011; Lange *et al*. 2014) as they share the same overall goal i.e., designing with quantifiable risk assessment of life safety and economic factors. However, there may be certain aspects that may need modification, as is evidenced by a detailed examination of structural fire engineering.

3.6 Probabilistic structural fire engineering

The current framework of probabilistic structural fire engineering (PSFE) is similar to PBEE as it effectively follows Equation 3.1, with the main difference being fire as the hazard. The interpretation of the PEER framework in PSFE is illustrated in Figure 3.3. Ideal pinch variables (IM, EDP, DM and DV) need to be identified to make the process robust. The process of PSFE begins with the identification of the fire hazard, λ(IM), which represents the annual rate of exceedance of IM. Examples of IM for fire engineering may include maximum fire temperature, total fire duration, area under the fire curve or cumulative radiant heat. A thermo-mechanical analysis of the structure exposed to the given fire scenario is required, in order to evaluate the likely performance of a structure in a probable fire. For most situations, this is achieved by a sequentially coupled analysis (i.e. thermal followed by structural analyses) using, where appropriate, suitable software, such as VULCAN (Huang *et al.* 2003a; 2003b, 2004), ABAQUS (ABAQUS, 2014), OpenSees (McKenna, 1997, Jiang J et al. 2015), ANSYS (ANSYS, 2012) or SAFIR (Franssen, 2011). EDPs (e.g. maximum displacement) are recorded during the analysis. For timber structures, it is becoming increasingly apparent that the fully coupled thermo-mechanical analysis is more appropriate.

Following the structural analysis, the damage to a structure is quantified based on the value of the EDP. The parameter chosen as the EDP needs to have a good correlation with key damage milestones, such as no or minor damage, major damage and severe damage or collapse. Figure 3.3 also illustrates a pictorial representation of fragility functions which define a probable level of damage based on structural response. Three damage states; d_{s1} (i.e. minor repair), d_{s2} (i.e. major repair) and d_{s3} (i.e. irreparable damage or replace) are generated based on the structural response. Consequence functions (probabilistic relationships between the damage states and loss/downtime) can then be used to estimate the annual impact of a fire hazard in terms of the performance measure of interest (e.g. failure, repair cost, downtime).

As the hazards (earthquake and fire) are similar (low probability-high consequence) in nature the PBEE framework has so far been directly applied in PSFE. However, research shows that there are problems with this direct application, as the quantification of the two hazards and

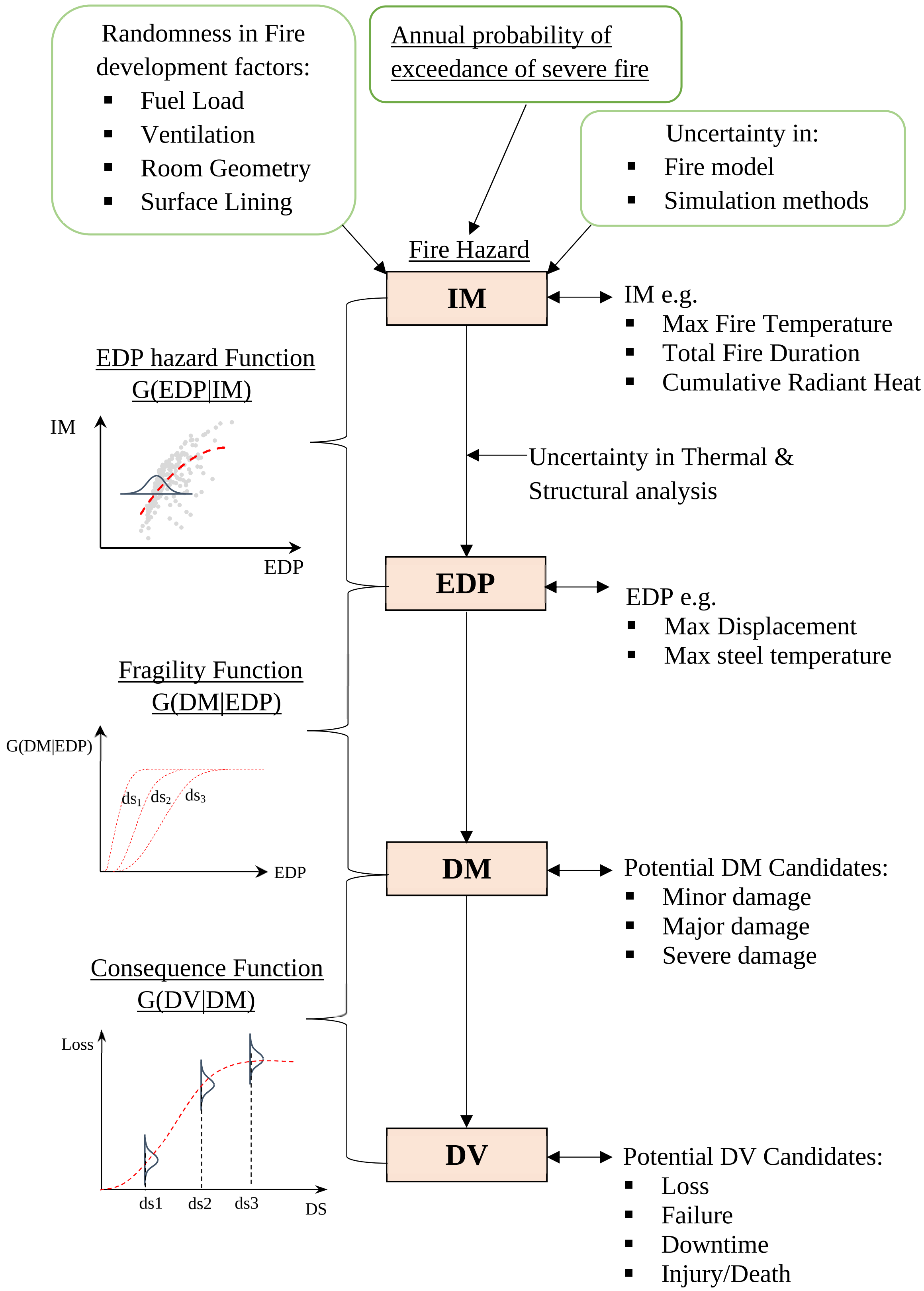


Figure 3.3: Flowchart illustrating components of the PSFE framework.

their associated structural response are different (Moss *et al.* 2014; 2016). The hazards in seismic engineering occur outside the building and manifest themselves in motions of the entire building, and structural response is only at room temperature. In fire the hazard may occur in one compartment within the building, and its detrimental effects require structural response in both thermal and mechanical terms. The temperature of the compartment is dependent on the geometry of the compartment itself, the fuel, ventilation conditions and the surface lining of the compartment. If surface linings absorb more heat, then the temperature rise within the compartment will be less intense. This temperature-time relationship may be obtained by simple or complex calculations. Simple calculations employ unique thermal inertia of each lining while a complex model will monitor thermal exchanges between each lining and the compartment throughout the fire. Thus, the thermo-mechanical response of the building in a fire is more complex than the more straightforward mechanical analysis in earthquake engineering. As a result of these fundamental differences in the hazard and associated building response, it has been observed that although the broad framework of PBEE is still applicable to PSFE, it needs to be investigated in detail to produce meaningful results for fire engineering design (Shrivastava *et al.,* 2018). In adopting the earthquake engineering methodology in structural fire engineering, it needs to be borne in mind that the backgrounds to terminologies and processes differ between disciplines. As such PSFE may need some modification to suit its purposes. Therefore, proposals to modify PSFE framework are discussed in the next section.

3.7 Modified Probabilistic structural fire engineering

The first stage of PSFE is referred to as fire hazard analysis in order to highlight the type of hazard for which the analysis is performed and also to differentiate it from PBEE. In earthquake engineering, the hazard happens outside the building and therefore its distinct amplitude (i.e. intensity) at the location of the building is very important because intensity reduces with distance, ground condition and magnitude of the earthquake. In fire the hazard happens inside the building. The severity of the fire is therefore affected by how much ventilation is available, the amount of combustible material available and the thermo-physical properties of the lining material. Intensity of a fire typically refers to the temperature of the fire at a given time. An intense short duration fire has less effect on a protected steel beam or concrete beam than a less intense long duration fire. Therefore, it is not the “intensity” of the fire which affects the structure but its overall severity. Therefore, this thesis proposes to call it a Fire Severity Measure (FSM) instead of an Intensity Measure (IM). FSM apprehends the significant characteristics of the fire scenario which affect the response of the structural system. Examples of FSMs are maximum fire temperature, fire duration, area under the fire curve and cumulative radiant heat, among others. Furthermore, in PBEE, the structural response is calculated for a given hazard, i.e. PGA, by performing only (room-temperature) structural analysis whereas in PSFE, the hazard is represented by a temperature-time curve. Once you generate the hazard then a two-phase analysis is required to get the complete response of the structure. First is a thermal analysis which estimates the temperature history of the structure for the given fire profile. Secondly, a mechanical analysis is performed to evaluate the response of the thermally affected member to the applied loads. Keeping the same terminology, i.e. structural analysis, in PSFE does not clearly capture the inclusion of thermal effects on the structure. Therefore, the thesis suggests the use of "response analysis" in order to clearly account for both thermal and structural analyses effects on the structure. It is important to note that depending on the jurisdiction "structural design" may involve some consideration of fire effects as part of the design, while other jurisdictions may treat the fires themselves under a completely different discipline. It is for this reason that the thesis suggests a change from "structural analysis" to "response analysis". Critical response parameters i.e. EDPs are identified at the response analysis stage. Some examples of structural response parameters (EDPs) are maximum displacement, maximum member temperature and maximum moment. The EDP needs to be correlated with a damage measure (DM), such as no damage, spalling, collapse, etc., which

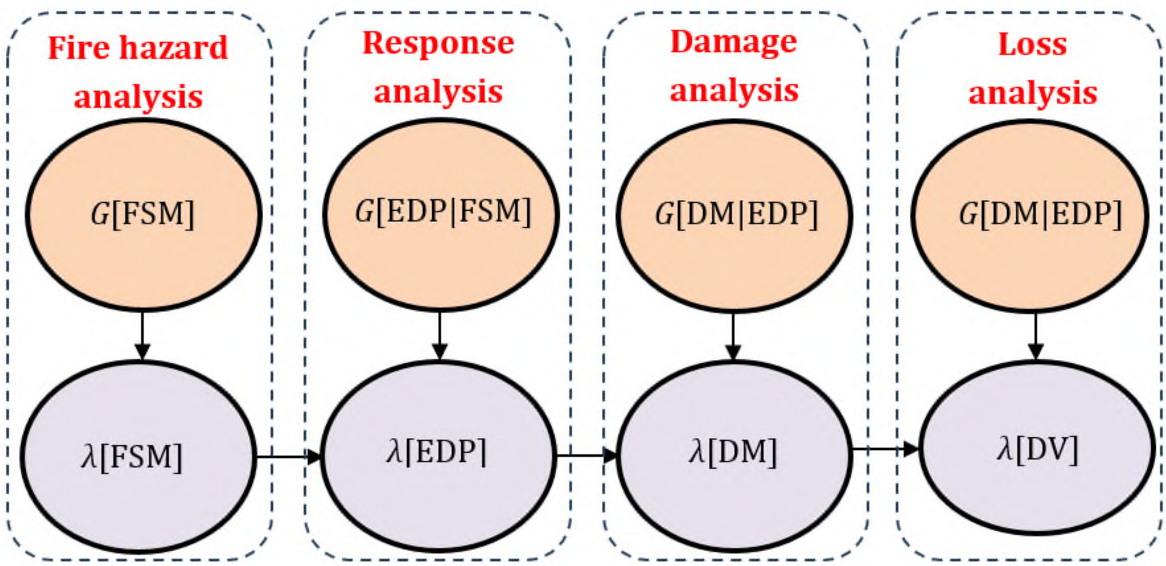


Figure 3.4: Probabilistic Structural Fire Engineering Framework

will be expressive of loss/cost. The adoption of the PBEE approach in PSFE is illustrated in Figure 3.4.

The PEER center equation based on the total probability theorem is now modified for PSFE (Equation 3.2). All the parameters in Equation 3.2 have the same properties as those in Equation 3.1 except the change in name of a variable:

$$\lambda(DV)=\iiint G(DV|DM) \mid dG(DM|EDP) \mid dG(EDP|FSM) \mid \lambda(FSM) \quad (3.2)$$

A significant proportion of the uncertainty in the fire response of a structure is due to the uncertainty in the characteristics of future fire events. This uncertainty can be handled by a probabilistic analysis, unlike a deterministic analysis. Evaluation of the uncertainty in the structural response is an important step in PSFE, whose objective is to meet the performance criteria. The uncertainty in estimating the severity of fire hazard in a structure is modelled by considering the Fire Severity Measure (FSM) as a random variable and evaluating the Mean Annual Rate of Exceedance (MARE) of the fire hazard at the site, λ(FSM). The uncertainty in estimating the structural response is incorporated by estimating the conditional probability of exceeding a value of the engineering demand parameter, given a fire severity measure P(EDP|FSM). Since DV is the final variable needed to estimate the performance of a structure, it is first necessary to evaluate the structural response for various fire scenarios of different intensities. In the fire hazard analysis, the rate of occurrence of a fire hazard with a range of fire severities can be calculated. Then the rates of occurrence of fire hazard can be coupled with the estimated structural response to compute the annual rate of exceedance of a given level of structural response (see Figure 3.5).

The focus of the current research is on FSMs and EDPs, as EDPs help to define FSMs appropriately. DMs and DVs are not specifically discussed in this research. A major outcome of the structural analysis is the estimation of EDPs, and with the help of some modification in the PEER framework equation (Equation 3.2), the mean annual probability of an EDP can be evaluated by integrating the conditional probability of P(EDP|FSM) for the given range of FSMs with FSM hazard as shown in Equation 3.3.

$$\lambda(EDP)=\int P(EDP > edp \mid FSM) \, . \, d\lambda(FSM) \quad (3.3)$$

Identifying a fire by a single characteristic may completely ignore other aspects of the fire. For example, a “two-hour long fire” does not give any information about the peak temperature of the fire or how quickly it decays. Furthermore, if the severity of fire hazard has to be represented by a single parameter (i.e. by an FSM) then it needs to be ascertained that the parameter used adequately accounts for the full properties of the fire more efficiently than any other parameter. In parallel to PBEE, a suitable FSM is identified by several selection criteria such as efficiency, sufficiency, scaling robustness, hazard computability and predictability. The application of all these properties to select a suitable FSM leads to an accurate prediction of the structural response. So far, only the efficiency criterion has been investigated in PSFE (Moss *et al.* 2014; 2016). Therefore, it is important to introduce more selection criteria in PSFE to produce suitable FSMs.

A few FSMs have been explored in the literature: maximum fire temperature, fire duration, area under the time-temperature curve and cumulative radiant heat (Moss *et al.* 2014; 2016). Maximum steel temperature is also used as an FSM by Hamilton. Principally, maximum steel temperature is a thermal response of the structure, which is a good indicator of steel structural performance in fire. Maximum steel temperature can be argued to be unsuitable as a fire severity measure (FSM), as FSM candidates need to be independent of the structure and should

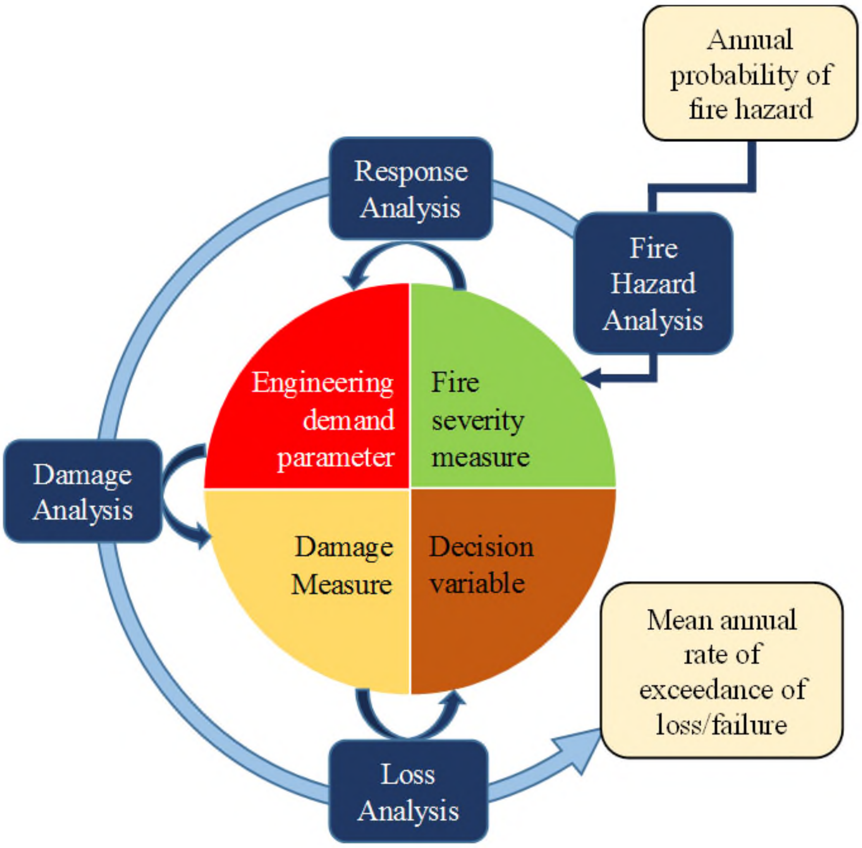


Figure 3.5: Probabilistic Structural Fire Engineering approach

be evaluated from the fire hazard curves or parameters contributing to the temperature-time curve. This indicates that very limited work has been done in investigating potential FSMs. Previous work to identify efficient FSMs was performed with a small number of alternatives, which is insufficient for a sufficiently appropriate estimation of structural response.

3.8 Fire Severity Measures

The temperature-time curves shown in Figure 2.4 are representations of some of the potential fires of the compartment. The severity of any hazardous event can be represented by many parameters, each of these being a potential severity measure. An important task of PSFE is to make a suitable choice of FSM. In order to fully understand which FSM represents the fire severity most precisely, this research investigates a wide range of FSMs as discussed below. The FSMs of interest are presented in Figure 3.6 and Figure 3.7 with the help of a temperature-time profile generated using the formulation of the Eurocode parametric fire. Traditionally, fire temperature is the most commonly accepted indicator of the fire severity. Therefore, it is preferred to use the temperature-time curve to represent the hazard. Fire severity measures are intended to be the best attributes to effectively represent the effects of the fire on the structure.

- Maximum Fire Temperature (MFT): this is the maximum temperature of the fire inside the compartment, as defined using the Eurocode parametric fire curve. The temperature of the fire affects the temperature of a structural member which influences the overall stability of the structure.

- Time to Maximum Fire Temperature (TMFT): this is the time at which the temperature in a compartment reaches its maximum value. It represents the duration of the heating phase of a fire.

- Fire Duration (FD): the fire duration is the total time taken by the fire to cool down to ambient temperature. It reflects the total exposure time of a structure to a fire event.

- Area under the fire temperature-time curve (AUC): this provides some information about the potential heat energy of a fire that a structure is exposed to. It is calculated as the area under the time-temperature curve. Unfortunately, this product does not have a physical meaning, but it is indicative of the effect the shape of the fire has on the structural response.

- Cumulative Incident Radiation (CIR): This is the total incident radiant heat flux to which a structure is exposed. The cumulative incident radiation will only consist of the radiative contribution of the fire (excludes the convective part). For spaces that are assumed to have uniform temperatures in the entire compartment the radiative and convective temperatures can be assumed to be equal to the gas temperature. CIR is calculated as the area under the incident radiant heat flux-time curve. It represents the amount of radiant heat that the structure is exposed to during a particular fire. The incident radiation is calculated from Equation 3.4 and is illustrated in Figure 3.7.

$$h_{rad} = \phi.\varepsilon.\sigma.(\theta_r + 273)^4 \tag{3.4}$$

h_{rad} (W/m^2) is the incident radiation heat flux at time t, ϕ is the configuration factor, θ_r is the radiation temperature in °*C* (here it is gas temperature due to the member being fully engulfed in fire), ε and σ represent the total emissivity and the Stephan Boltzmann constant (5.67 x 10^{-8} W/m^{2}K^4) respectively.

- Fire load density per floor area (q_{fd}): this is the fire load density related to the floor area of the compartment. The fuel loads of the compartment show the amount of potential heat that can be released during the fire event.

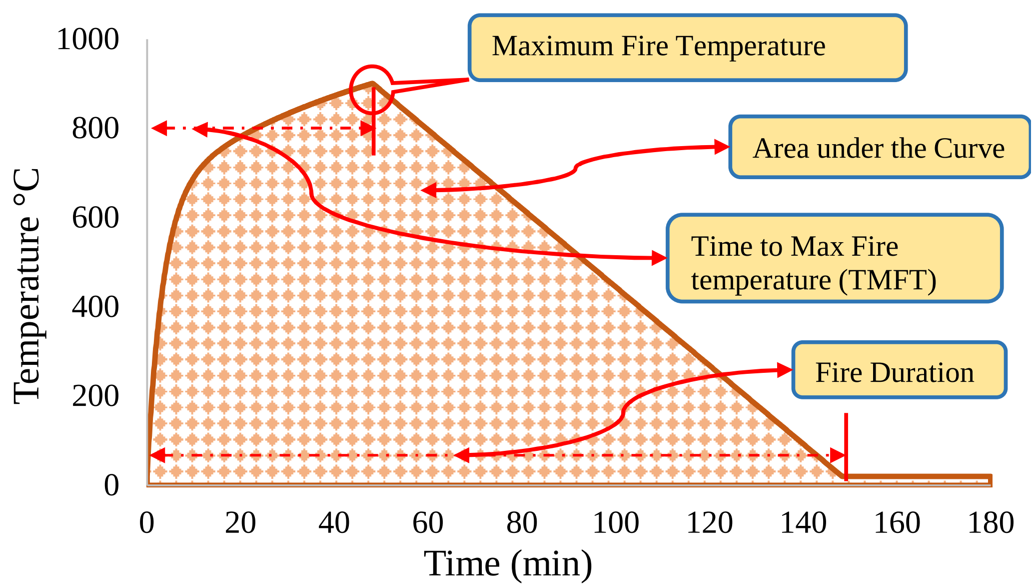


Figure 3.6: Temperature-Time profile.

- Fire load density per internal surface area (q_{td}): this is similar to the fire load density per floor area, but this time related to the total internal surface area of the compartment.\

Precise quantification of FSM is necessary to accurately predict the structural response. Detailed explanations of selection criteria for the identification of a suitable FSMs are presented below.

3.9 Selection criteria of FSMs.

The implementation of FSM selection criteria in PSFE is discussed here. Efficiency (Shome and Cornell, 1999), Sufficiency (Luco and Cornell, 2007), Scaling robustness (Tothong and Luco, 2007), Hazard computability (Giovenale *et al.* 2004) and Predictability (Kramer and Mitchell, 2006; Tothong and Cornell, 2007) are well established in PBEE and the application of all these properties would lead to an accurate prediction of structural response in structural fire engineering.

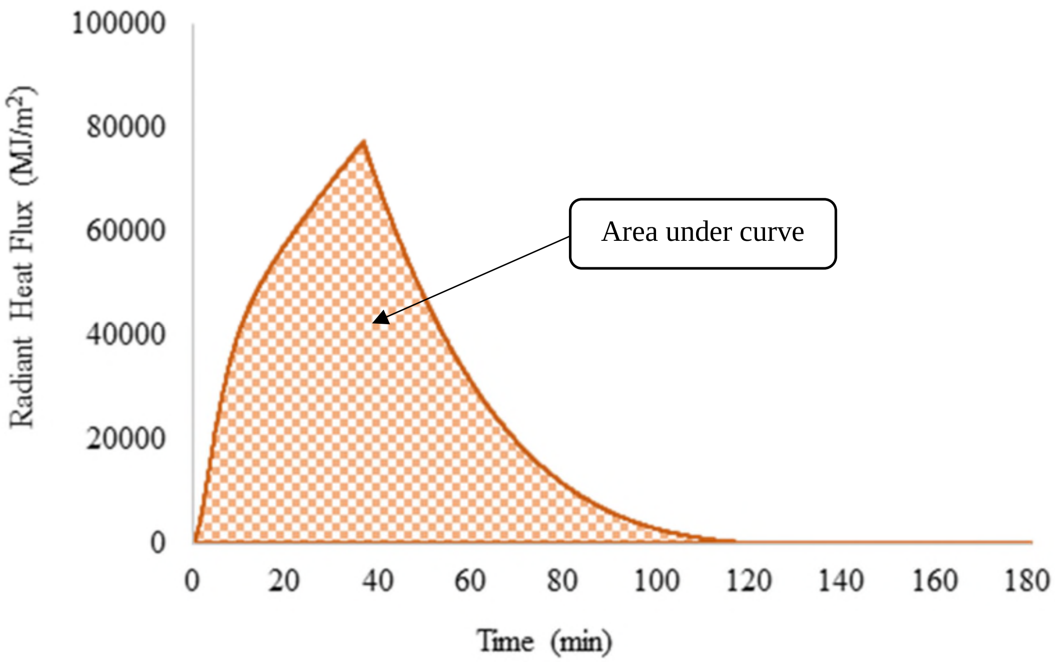


Figure 3.7: Radiant heat Flux vs Time

Efficiency

Efficiency of a FSM relates to the number of analyses required to achieve the requisite level of confidence in estimating the structural response. As discussed in Section 3.2, an efficient FSM yields a relatively small variation in the predicted response of a structure exposed to different fire events of the same intensity. In PBEE, response data (EDP) have been found to conform to a lognormal distribution. Therefore, efficiency is measured as the dispersion of the structural response. This concept is directly applicable in PSFE to find an efficient FSM. Moss *et al.* (2014) have so far successfully implemented the efficiency criteria in their studies to identify a number of potentially suitable FSMs.

Efficiency gives a measure of correlation of FSM with EDP. An efficient FSM provides a relatively smaller dispersion of the response of the structure. It is a measure of variance in the EDP-FSM relationship. The smaller the dispersion, the better the efficiency of the FSM. A better FSM, therefore, reduces the number of analyses required to achieve a desired level of confidence in the response of the EDP (Mackie and Stojadinović, 2003).

Sufficiency

A FSM is called sufficient when the structural response (i.e. EDP) for a given hazard (i.e. FSM) is independent of the variations in the key elements of the fire hazard. In other words, it specifies that the precise selection of records is not significant and the response of the structure will be accurately estimated irrespective of the variation in key elements of the fire hazard of the selected fire records, provided the same magnitude of fire severity measure is obtained. Thus a sufficient FSM would not distinguish between a short duration high-temperature fire and a long-duration low-temperature fire if they produce the same structural response. Sufficiency is quantified by measuring the trend of the residual plot. No specific trend in the residual plot, i.e. no error in the prediction by the regression model with respect to the computed EDP, as a function of other fire input parameters indicates the sufficiency of the FSM chosen.

Scaling Robustness

In PBEE, scaling of ground motion records is common to push the structure from no damage to the collapse state. Since extensive scaling of fire records, to cover low intensity to high intensity level produces unrealistic fire scenarios, therefore extensive scaling is avoided in PSFE. Scaled fire records and unscaled fire records of similar FSM value should produce similar structural response ideally to have a robust scaling process.

Hazard Computability

Hazard computability is defined as the level of effort required to quantify/generate fire hazard curves in terms of the proposed FSM. In PSFE, the evaluation of maximum fire temperature or fire duration is less intricate in comparison to cumulative radiant heat. Therefore, the effort required to represent any hazard with a FSM is also an important selection criteria for suitable FSM.

Predictability

Predictability (Kramer and Mitchell, 2006) is a measure of the accuracy with which a FSM can be predicted. If the predictability of an FSM is poor, then the accuracy in predicting the fire response (for a given fire scenario) will also be poor. The other selection criteria, such as practicality (Padgett *et al.* 2008) effectiveness and proficiency, require more extensive research to understand their applicability in PSFE. An application of all these selection criteria in PSFE will deliver an ideal FSM that enables the precise quantification of structural response and damage. Therefore, in order to accurately evaluate the annual probability of loss (or any other DV) FSM should satisfy the aforementioned properties.

Maximum Fire Temperature (MFT) has been commonly used by researchers as the FSM to represent fire severity (Lange *et al.* 2014; Moss *et al.* 2014). Moss *et al.* (2014) have investigated the efficiency criteria and this is discussed in a later section. The suitability of potential fire severity measures such as total fire duration, cumulative radiant heat, area under fire curve and MFT were all investigated. Cumulative radiant heat was found to be an efficient FSM for concrete beams for the estimation of EDPs, i.e. maximum displacement (Moss *et al.* 2014). Hamilton, (2011) considered maximum steel temperature (MST) as an FSM. Principally, MST is a thermal response of the structure to the fire loading and can be argued to be unsuitable as an FSM candidate needs to be independent of the structure and should be evaluated from the fire hazard curves or parameters contributing to the fire temperature curve.

EDPs

As discussed in Section 3.3, Engineering Demand Parameters (EDPs) are the response parameters of the structure, which can be used to estimate damage to structural components and system. A number of researchers have explored various EDPs in PBEE that could be used to improve performance prediction and reliability. On the other hand, in PSFE there is very limited work so far on identifying EDPs. The EDPs identified so far are maximum deflection, maximum steel temperature, spalling, residual capacity and peak rebar temperature (Lange *et al.* 2014; Moss *et al.* 2014; Moss *et al.* 2016; Rush & Lange, 2017; Iounnou *et al.* 2017; Rush *et al.* 2014). Some structural response parameters that may be considered as potential EDPs are maximum bending moment and lateral displacement of columns. An ideal EDP correlates best with damage to the structure at the performance level (Whittaker *et al.* 2004).

3.10 Analysis methods

Analysis methods are used to establish the relationship between FSM and EDP, which can subsequently facilitate computation of annual probability of exceedance of structural response. Structural responses are estimated for given hazard severity levels and then the results are processed with analysis methods to predict the probabilistic response of a structure. To obtain this prediction and establish the relationship between EDP and FSM there are several methods available in PBEE whose extension to PSFE may be useful. These are Single Stripe Analysis [SSA] (Jalayer, 2003), Multi Stripe Analysis [MSA] (Baker, 2007; Bazzurro *et al.* 1998; Jalayer and Cornell, 2009), Incremental Dynamic Analysis [IDA] (Vamvatsikos and Cornell, 2002) and Cloud Analysis [CA] (Bazzurro *et al.* 1998; Jalayer, 2003; Luco and Cornell, 2007), as mentioned in Section 3.5. These methods are fairly well developed and are used based on requirements of the level of accuracy and desired computational effort in estimating the FSM and EDP relationship. Each method has its significance and has the ability to relate earthquake hazard intensity (IM) with structural response (EDP) in its own way. The analysis methods were discussed in detail in Section 3.5

There is limited literature available on the use of analysis methods to demonstrate the relationship between EDP and FSM in PSFE. Moss *et al.* (2014) conceived a method called Incremental Fire Analysis (IFA). IFA is analogous to IDA in earthquake engineering and initially proposed to perform extensive scaling of fire profiles to generate sufficient data points at each FSM level.

3.10.1 Incremental Fire Analysis (IFA)

The IFA process begins with the generation of a limited number of temperature-time profiles using different values of fuel load, ventilation, surface lining and room geometry. Thereafter, in order to have a catalogue of fire profiles, these fire profiles are scaled to various targeted FSM levels. For example, if Maximum fire temperature (MFT) is chosen as the FSM, Figure 3.8 demonstrates scaling of fire profiles to a maximum fire temperature of 913°C. Fire profiles which have peak temperature less than or greater than 913°C are scaled to reach a peak temperature of 913°C using a single scaling factor for both the burning and cooling phase of each fire. The scaling factor is different for each fire profile to get fire to desired maximum fire temperature.

Similarly, if total fire duration (TFD) was chosen as a FSM, the records could be scaled to have the same FSM value e.g. 122 minutes in Figure 3.9. The unrestricted scaling of fire profiles over such a wide range may distort other characteristics of the fires such as time to reach maximum fire temperature, area under the fire curve and peak temperature. For example, a fire with a high peak temperature is expected to have a short duration and vice versa. Thus a scaling methodology that only varies the maximum temperature without commensurate adjustment of duration is not adequately characterising the fire hazard. Therefore, an amendment in the existing Incremental fire analysis is required to prevent inappropriate scaling of fire profiles which is discussed in Chapter 4.

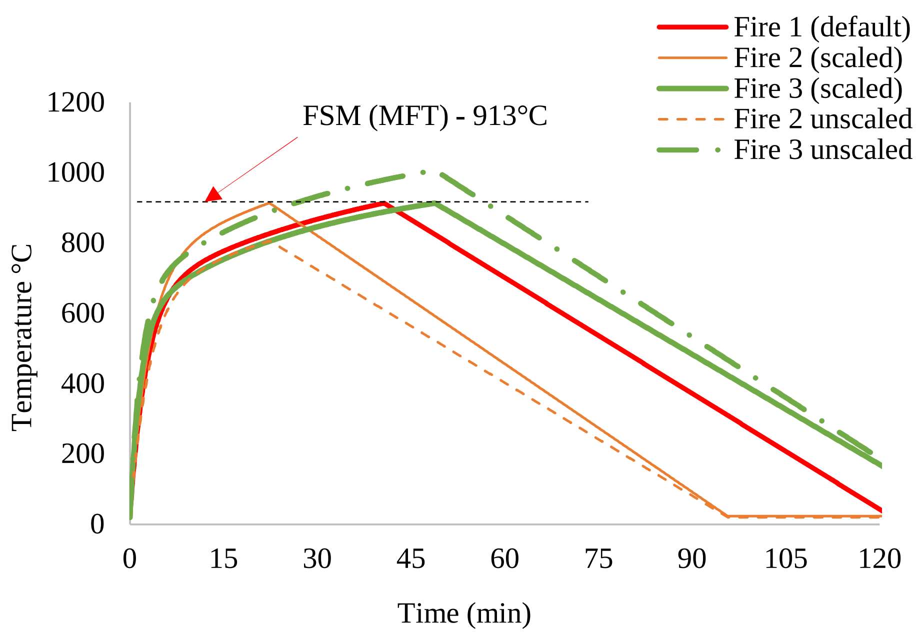


Figure 3.8: Scaling of fire profiles to Maximum fire temperature of 913°C

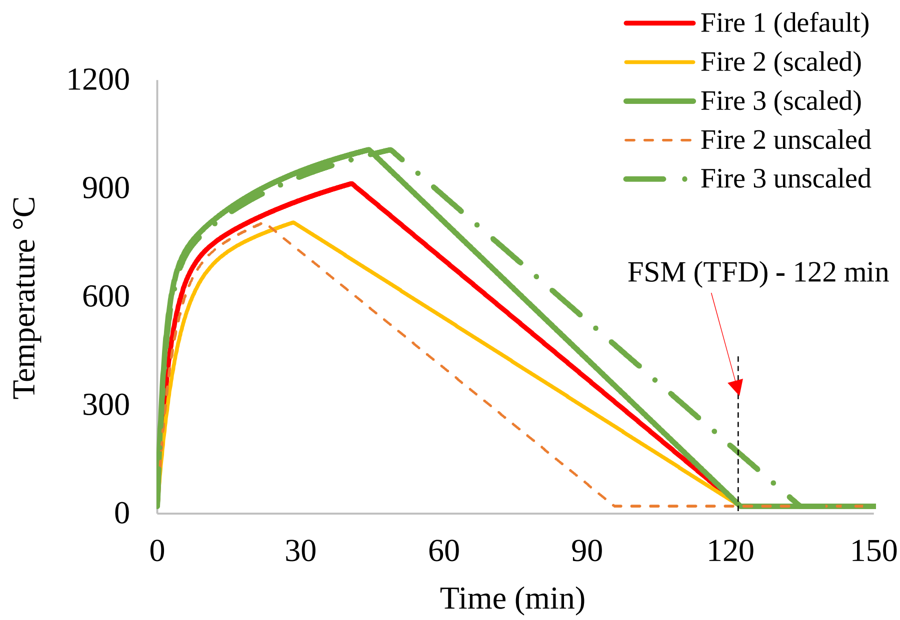


Figure 3.9: Scaling of fire profiles to Fire duration at 122min

Chapter 2 presented a comprehensive summary of structural fire engineering concepts. The basis of the probabilistic structural fire engineering framework (i.e. PBEE approach) was discussed in detail in Chapter 3. Modifications of various aspects of the PBEE approach to its application in PSFE was proposed wherever required to suit the context of structural fire engineering. The drawbacks of earthquake engineering analysis methods to establish relationships between fire hazard and structural response were also discussed. Chapter 4 address this issue by the implementation of two new analysis methods in PSFE. Chapter 5 then identifies the most suitable parameter to quantify fire hazards for structural response. Chapter 3 identified various sources of uncertainties in PSFE. Uncertainties related to structural modelling are investigated in Chapter 6, while uncertainties on the effects of fire models on structural response are covered in Chapter 7. The literature review also identified that currently available mean value of fuel load, ventilation, room geometry and surface lining are based on old surveys. Therefore, the research also proposes distributions for the post-flashover fire development factors (see Chapter 7).

4 FIRE STRIPE ANALYSIS AND CLOUD ANALYSIS

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2016. Efficiency of different intensity measures for probabilistic fire engineering. In: *Australasian Conference on Mechanics of Structures and Materials* XXIV (ACMSM24). CRC Press, 2016. Pp 707-712.

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2017. Analysis methods in probabilistic structural fire engineering. In: *Proceedings of the 2nd International conference on structural safety under fire and blast loading (CONFAB 2017)*: 278-278

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2018. Severity Measures and Stripe Analysis for Probabilistic Structural Fire Engineering, *Fire Technology, p. 1–27*

Shrivastava M., Abu A. K., Dhakal R. P., Moss P. J., and Yeow T. Z., 2018. "Probabilistic structural fire design using incremental fire analysis and cloud analysis", *Proceedings of the Institution of Civil Engineers - Engineering and Computational Mechanics*, Vol. 173, Issue 1, pp 11–29.

Chapter 3 discussed the various aspects of the PSFE approach. It also proposed modifications in PSFE to represent the structural fire engineering terminologies better. The drawbacks of the analysis method IFA were highlighted and the need for new analysis methods was argued. An alternate approach to IFA could be to group fire profiles in bands/bins based on their FSMs. The profiles within each bin are then scaled to the mean/fixed value of the bin. This approach restricts the size of the scaling factor, and results in scaled fire profiles which are more realistic. Instead of scaling of a handful of temperature-time profiles, a suite of fires can be generated based on probabilistic distributions of post-flashover fire development factors such as fuel load, ventilation, thermal inertia of the compartment and room geometry. Therefore, this chapter introduces a new method called Fire Stripe Analysis (FSA), to avoid extensive scaling of fire profiles as is associated with IFA, to help establish relationships between FSMs and EDPs. This chapter demonstrates the application of the PEER approach in structural fire engineering through an example performed on steel-concrete composite beam element and demonstrate the application of analyses methods. The chapter also demonstrates how the Cloud Analysis method from PBEE may be applied to PSFE.

4.1 Fire Stripe Analysis (FSA) Overview

The general procedure demonstrating the methodology and application of FSA is illustrated through a flowchart in Figure 4.1.

The probabilistic distribution of several post-flashover fire development factors generates several input values which can be used directly in a fire model (e.g. Eurocode parametric fire model) to produce temperature-time profiles, from which the various FSM values are derived. To aid the explanation of the process maximum fire temperature (MFT) is chosen as FSM. The maximum fire temperature values are calculated for each fire profile.

In FSA, FSM levels are carefully chosen to cover a wide range of severity of fires which apprehend the significant response of the structure, and hence signify damage.

There are two ways through which FSM levels are selected; by an “equal intervals” or “equal data points” approach. In the ”equal intervals” approach, bins are defined considering constant increments of FSM, and fire profiles whose FSM values fall within that band are grouped within it. These grouped fire profiles are scaled to the central FSM value of each bin. In the ”equal data points” approach, bins are defined considering an equal number of FSM values, and an equal number of fire profiles within that band are grouped. These grouped fire profiles are scaled to the central FSM value of each bin.

Both “equal intervals” and “equal data points” approach, is explained with the help of an example. The example used maximum displacement as the engineering demand parameter (EDP), following research by Moss *et al.* 2014 and Devaney, 2014.

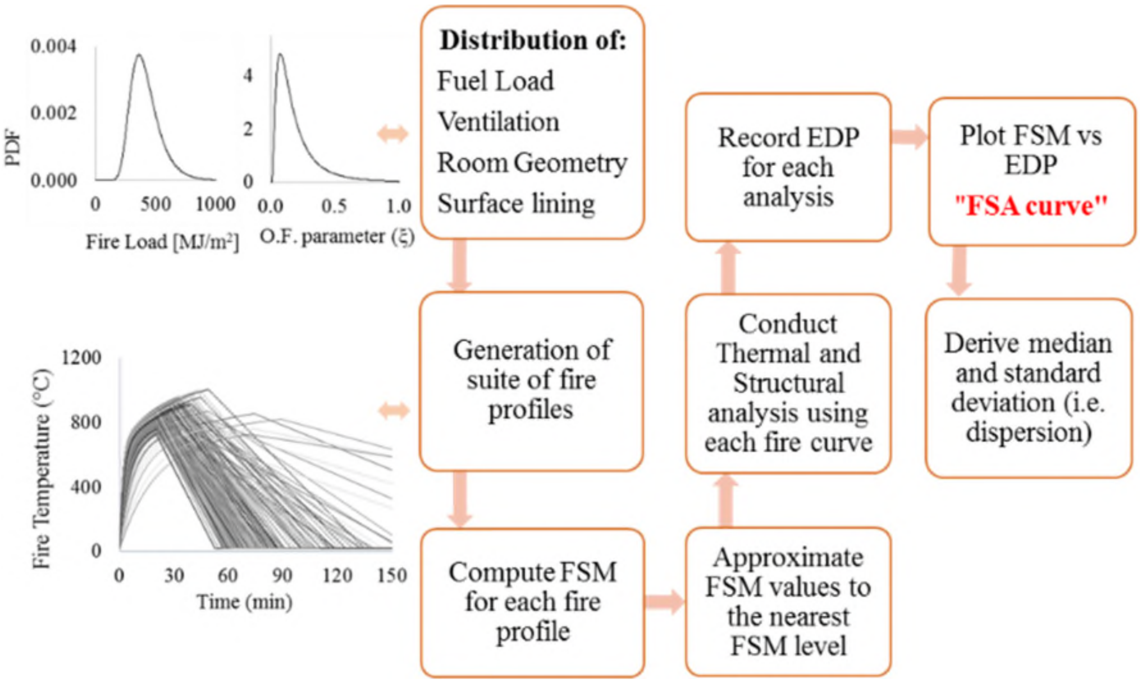


Figure 4.1: Fire Stripe Analysis (FSA) in Probabilistic Structural Fire Engineering

In order to evaluate structural response in the probability domain, a structure needs to be exposed to several possible fire profiles. In earthquake engineering, historical records are available for the analysis at a particular location or a location similar to a considered site. However, there is no data available for fire engineering based on the building structure, location and use. Fire and Emergency New Zealand (FENZ) does maintain records of fire occurrences but it has been observed that those records lack structural details such as compartment dimensions, ventilation, material of construction, etc. Therefore, an appropriate way to obtain a suite of fire profiles is to generate them randomly using distributions of input values for post-flashover fires. Post-flashover fires are used in the analysis because they represent the majority of fires that significantly affect the structure. Here the suits of fire profiles are generated using randomly varying fuel loads and ventilation of the compartment. There are other sources of uncertainty, such as randomness in surface lining, room geometry, fire models, structural models, thermal analysis approach, material uncertainty and many more. All these uncertainties have an impact on the evaluation of probable structural response. However, for the purpose of enabling probabilistic analysis and to focus on the response of the structure, variation in only two parameters was considered in this example.

4.2 Structure modelled

An office building constructed in New Zealand in 1988 was considered for the purpose of this research (Stevenson, 1993; Wastney, 2002; Welsh, 2001). Typical floor plan details are shown in Figure 4.2. It was a composite structure of steel beams and reinforced concrete slabs. The beam size was 610UB101 having full composite action with a 120 mm thick concrete slab (65 mm continuous depth and 55 mm decking height) as shown in Figure 4.3. The beam was located at the centre of the plan. The reinforcing steel used in the slab was A193 mesh (193mm^2/m) which was located at mid height of the continuous portion of the slab. The beam and slab were exposed to various time-temperature profiles on three sides. The gravity beam was modelled in the analysis using the finite element software VULCAN (Huang *et al.* 2003a; Huang *et al.* 2003b, Huang *et al.* 2004) as shown in Figure 4.4.

The compartment was 18 m by 20.5 m in floor area with a height of 3 m. Maximum ventilation area was taken as 15% of the floor area (54.74 m^2) with an average window height of 2 m. The maximum ventilation area was based on covering the range of fire profiles from short sharp fires to long cool fires. The walls and roof of the building were assumed to be made of normal weight concrete. The gravity load, a combination of dead and live loads, used in the analysis was uniformly distributed load along the length of the beam. Similar to the loading conditions of Stevenson (1993), the most adverse fire design load combination of 50 kN/m was used in this study. Geometrical and material parameters used in the analysis are shown in Table 4.1.

Table 4.1: Deterministic geometrical and material parameters

Parameters	Value	Units
Thermal Inertia (b)	1558.46	$J.s^{-2}/k/m$
Floor area	369	m^2
Total area	959	m^2
Maximum opening factor	0.08	$m^{1/2}$
Yield strength of steel beam	275	MPa
Yield strength of Rebar	500	MPa
Concrete strength	25	MPa

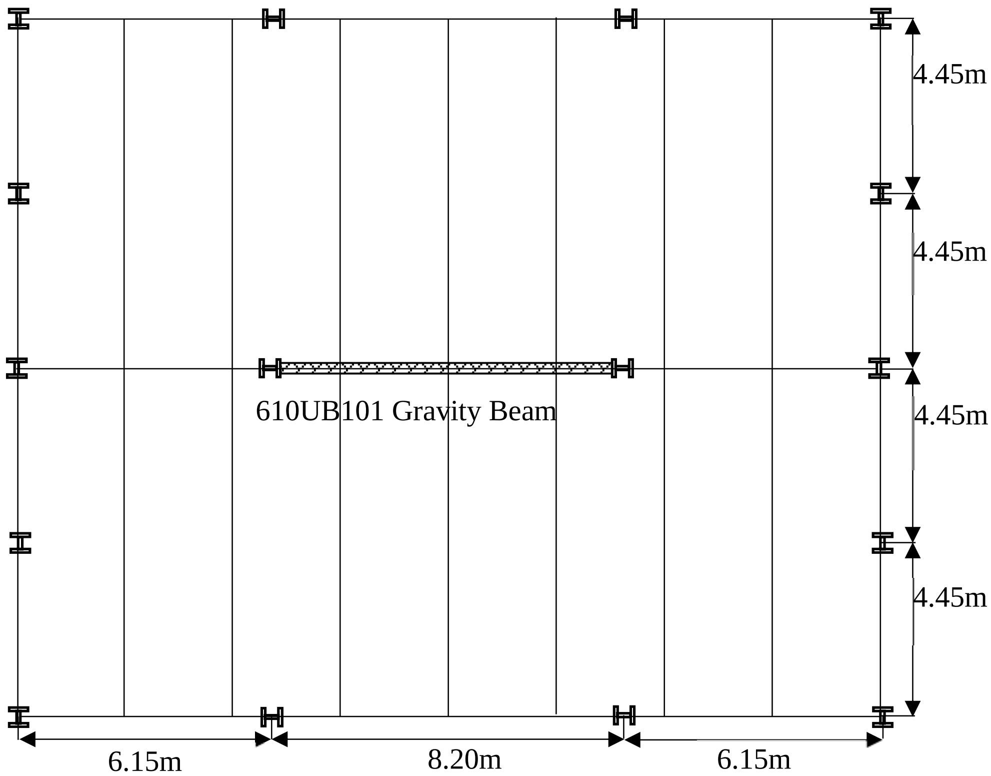


Figure 4.2: Floor plan of the building

Thermal and material properties of basic materials (i.e. steel and concrete) from the Eurocode are predefined in VULCAN. Therefore, thermal and material properties are not explicitly mentioned here. Thermal analysis of steel structure is guided by the method specified in the Eurocode and one-dimensional heat transfer was performed for concrete structure (Buchanan & Abu, 2017). A pin-pin support condition was used for modelling the composite beam. The ends were fully restrained against translation but not rotation. The edges of the concrete slab were restrained from moving perpendicular to the span of the beam and from rotating around their axes. These restraints were implemented to account for the effect of the adjacent structure. The pin-pin condition for the beam was chosen based on a preliminary study which produced the worst structural response, shown in Figure 4.5.

Figure 4.5 shows that if the ends of a beam are supported differently for the same loading condition then the pin-pin support condition may survive the exposure longer than other support conditions with large deflection. The results are similar to the research performed by Welsh (2001) which concluded that the pin-pin support condition provides ideal fire resistance due to thermal bowing and positive bending moment which allows the concrete slab to contribute to the flexural strength. In Welsh's research, the fixed-fixed support case also failed before any other support condition while the pin-roller support case showed runaway failure with a steep increase in deflection. The beams with supports that release moments (such as pin-pin or roller) show larger displacements in fire events due to the absence of restraining (hogging) moments at the supports. Welsh (2001) stated that the cause of the failure of the fixed-fixed support case was due to large stresses in the flanges at the ends of the beam, which occurred because of the thermally induced compression forces and negative bending moment. The yielding of the top flange was preceded by the bottom flange. The behaviour is found to be consistent with the Cardington Test where local buckling occurred in the bottom flange near the supports. The observation in Figure 4.5 is as a result of an instability failure in the software for the fixed-fixed case. VULCAN is not able to sufficiently track local buckling, and "stops" when global buckling occurs. In this particular case there is a large build up of compressive stresses in the beam cross-section, as a result of the increasing axial (compressive) force and hogging moment, which the programme could not sufficiently deal with. In tests, such as the restrained beam test at Cardington, it is observed that the bottom flange of the beam experiences local buckling, which in turn then allows the developmet of catenary action before an eventual failure of connections either at elevated temperatures or during cooling.

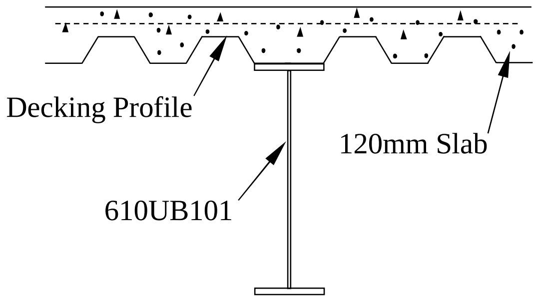


Figure 4.3: Cross-section of the beam

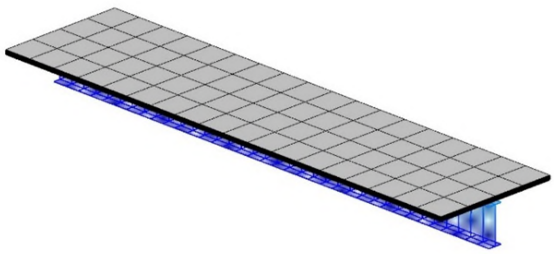


Figure 4.4: VULCAN model of composite beam and slab

Fire protection thickness of 16 mm was chosen based on the design of the beam for 60 minute fire rating using spray protection of 300 kg/m^3 density, specific heat capacity of 1050 J/kgK and thermal conductivity of 0.15 W/mK.

4.3 Generation of temperature-time profiles

As discussed in Section 2.2, the distribution of fuel load is obtained from Eurocode 1 which suggests a Gumbel distribution for fuel load in an office building with an average value of 420 MJ/m^2 and a coefficient of variation (COV) of 0.3 (CEN, 2002). This distribution was used to generate random values of fuel load from 200 to 1000 MJ/m^2. The maximum ventilation of the compartment was 54.74m^2. Based on the JCSS code (Vrouwenvelder, 1997) the availability of the opening factor varies log-normally as shown in Equation 2.2. Figure 4.6 shows the distribution of opening factor. However, the opening factors used in this study ranged from 0.02 to 0.08 m$^{1/2}$. With the help of the mean values and distribution functions of fuel load and opening factor, 200 fire records (i.e. temperature-time profiles) were produced with the help of Monte Carlo simulation (Figure 4.7). Stochastic variables are summarised in Table 4.2.

Table 4.2: Stocahstic variables used in the analysis

Parameters	Value	Variation	Distribution Type	Units
Fuel load	420 (mean)	0.3 (COV)	Gumbel	MJ/m^2
‘ξ’ Parameter for Opening factor	0.2 (mean)	0.2 (Standard deviation)	Truncated Log-normal	-

As can be seen in the time-temperature fire profiles, the maximum temperatures of the fires ranged between 600°C and 1050 °C. This was because of the low average value of fuel load provided by the Eurocode. Also, very limited short-sharp fires and long-cool fires were observed in Figure 4.7 because of the use of the distribution of available ventilation which generated a limited range of opening factors together with a low average fuel load. Suites of fire profiles have been generated by many researchers considering similar variation in fuel load and the ventilation but there are differences in geometry of the compartment.

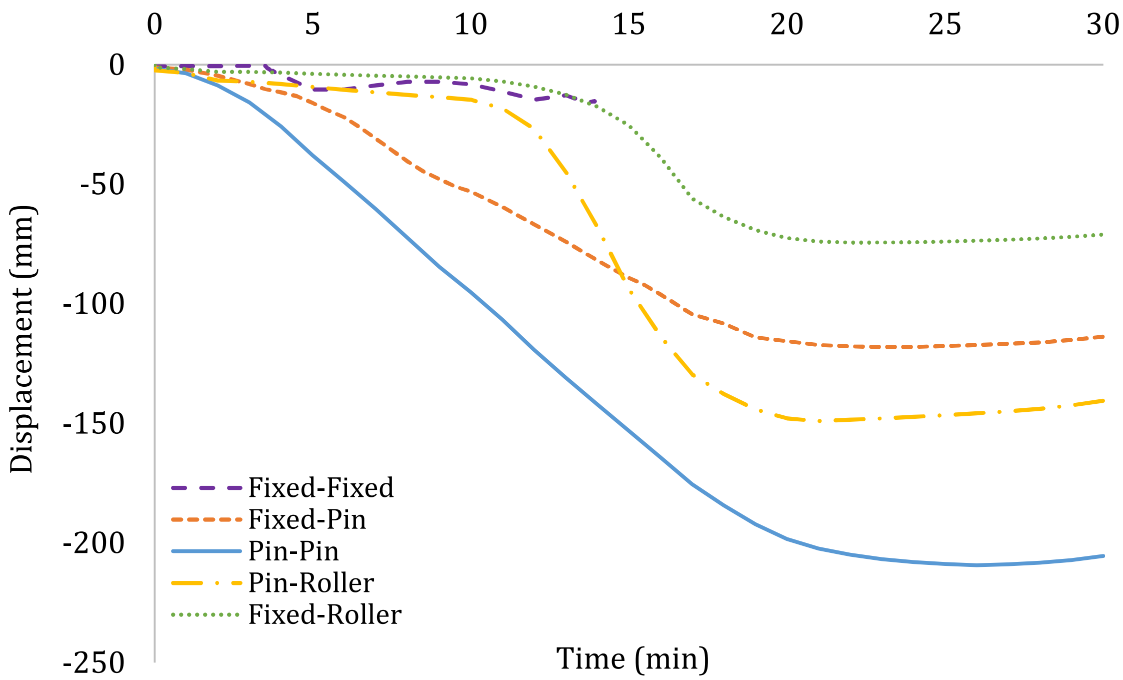


Figure 4.5: Comparison of deflection for different support condition

In the “equal interval” approach, fires were categorised in bands having equal intervals of severity measures. In order to explain the restricted scaling concept of FSA (see Figure 4.8) two fire profiles were picked from the suite of fire profiles. For illustration, a band of 50°C with a FSM level of 700°C is shown in Figure 4.8. Fire profiles having maximum temperature ranging between 675 to 725°C were scaled to a FSM level of 700°C. Fire-1 has MFT = 719°C and Fire-2 has MFT = 728°C, which were approximated to reach MFT of 725°C as illustrated in Figure 4.8. This restricted scaling of fires within the narrow bands is considered reasonable, as the amount of scaling is very small, and therefore it does not significantly affect the shape and properties of the fire curve. Similar minute scaling was performed at other FSM levels with the generated fires. Thermal and structural analyses were then performed for these modified fire profiles and a structural response parameter i.e. EDP, maximum displacement, was recorded for each analysis, see Figure 4.10.

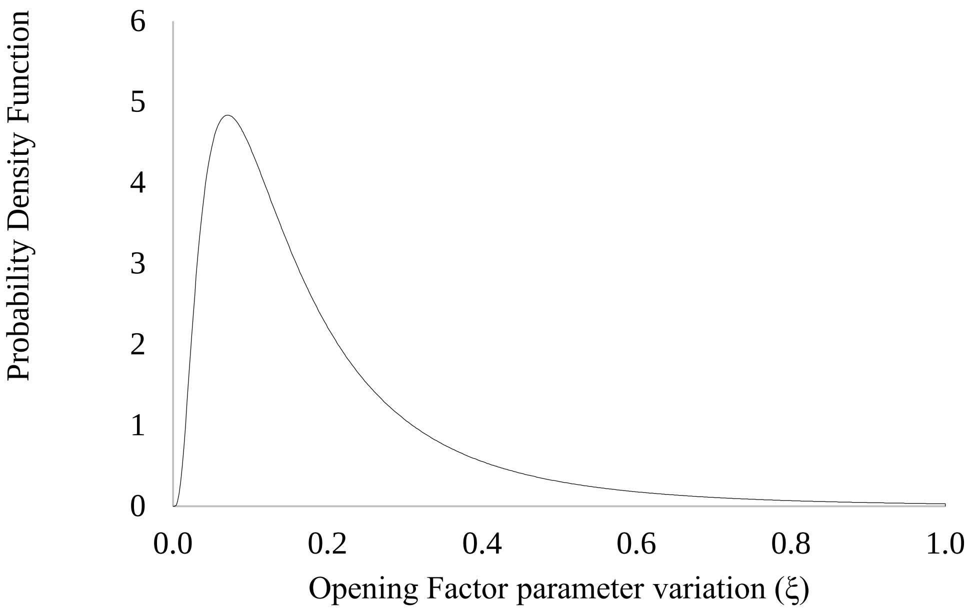


Figure 4.6: Probability Density function of opening factor (values ranging between 0.02 to 0.08 is considered in study)

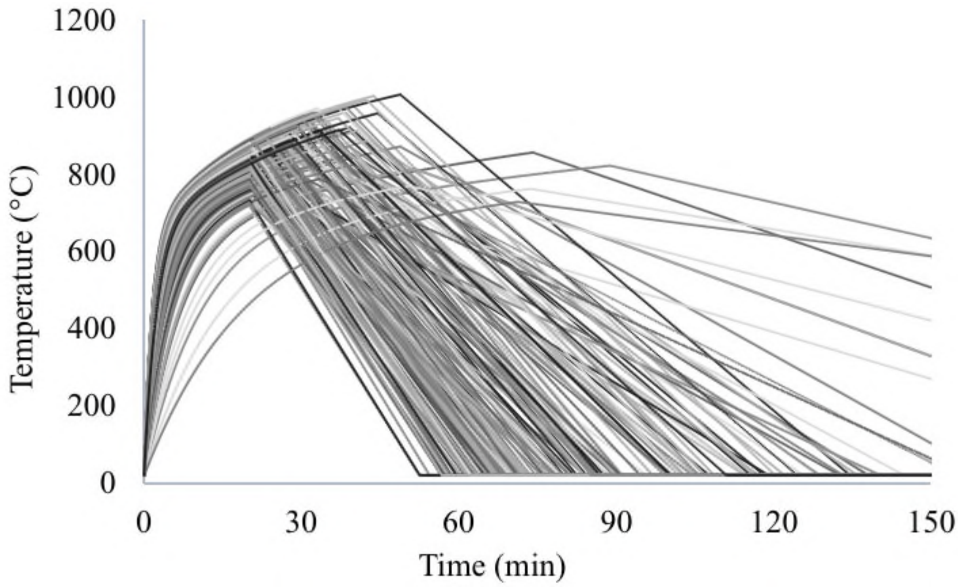


Figure 4.7: Temperature-time profiles

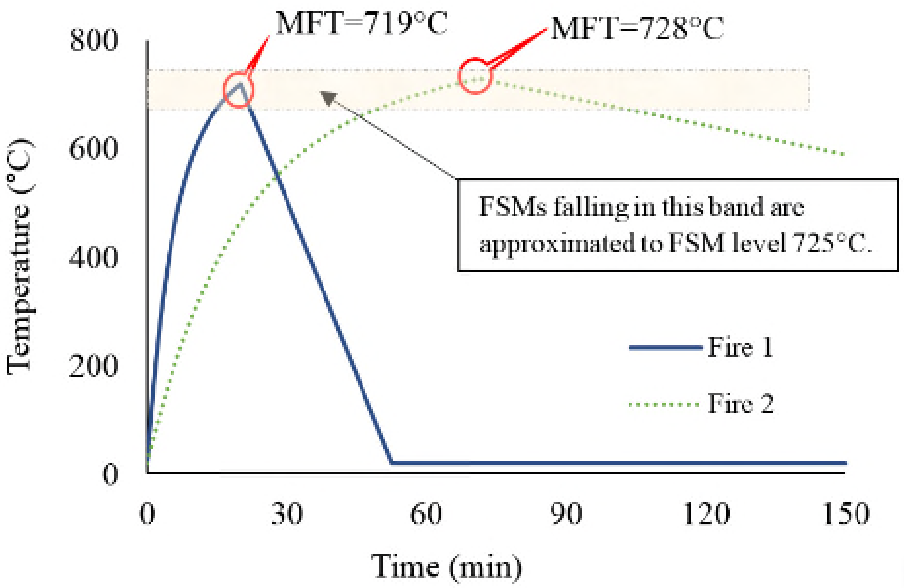


Figure 4.8: Fire profiles indicating FSM values and FSM levels

The results are comparable with the results of the research performed on a similar building by Moss & Cliffton (2002). The blue line in the curve in Figure 4.9 is the temperature profile used by Clifton and Moss and the red line is one of the temperature profiles used in this research.

Although there is some difference (10%) in the maximum temperature in the profiles, the amount of energy released during the heating regime is also proportional.

The maximum midspan deflection of the beam reported by Clifton and Moss was 340 mm for a similar gravity beam whereas the maximum mid span deflection in this research by using the above-mentioned temperature profile is 305 mm. The difference in the maximum deflection is proportional to the maximum fire temperature. Even though this situation is justifiable if it is an elastic stage, it appears that the deflection is increasing with a constant slope without runway failure in both analyses. The results of the research by Clifton and Moss were compared with Cardington test results and they found it satisfactory. Therefore, the results obtained from the structural model used in this thesis can be considered validated.

In the "equal data points" approach, the fire profiles are binned so that each bin has the same number of fire profiles. The fire profiles of each bin are then scaled to the mean FSM of all data points within each bin. 200 fires were generated in the example which created the data points of maximum displacement. If FSM range is divided into 20 bins, then in each bin we need 10 data points and these 10 profiles are scaled to the corresponding mean FSM levels as shown in Figure 4.11.

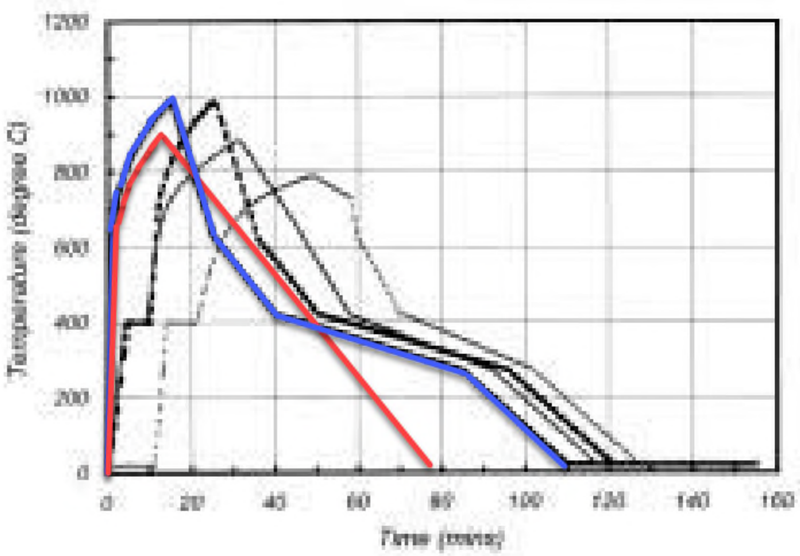


Figure 4.9: Comparison of a fire profile generated in this research with temperature profile in literature (Moss & Cliffton, 2002)

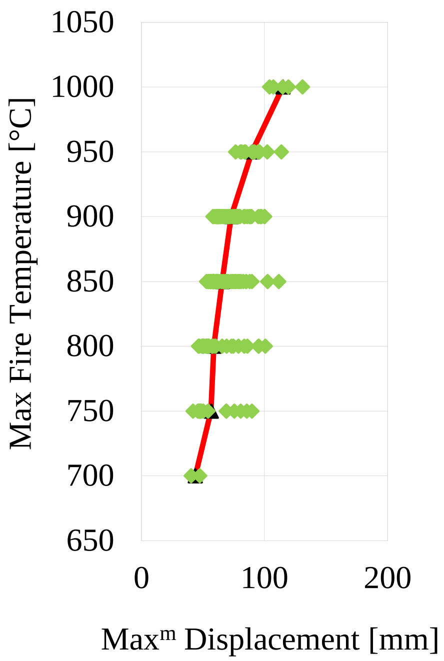


Figure 4.10: Fire Stripe Analysis plot Maximum fire temperature - Maximum displacement for “Equal interval approach”.

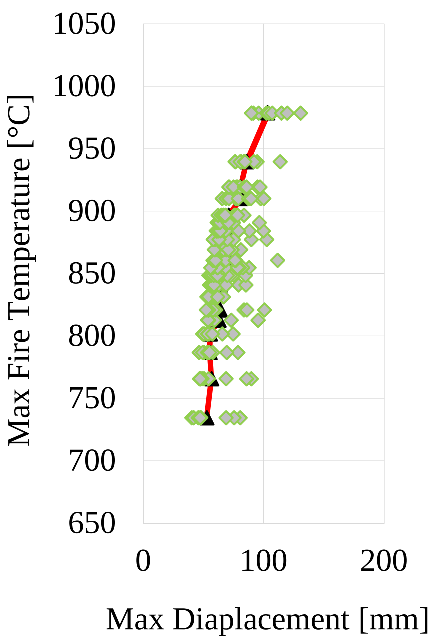


Figure 4.11: Fire Stripe Analysis plot Maximum fire temperature - Maximum displacement for “Equal data points approach”.

Each fire profile (or say each MFT value) produces one maximum displacement value, which is plotted as a point on a FSM-EDP (MFT-max displacement) graph. A group of EDP values is collected at each FSM level as shown in Figure 4.10 and 4.11; median and dispersion (assuming lognormal distribution) of the EDPs are estimated at each FSM level with the help of Equation 4.1 and 4.2 respectively. Research in PBEE has demonstrated that lognormal distribution fits the structural response, therefore lognormal distribution is assumed here in PSFE.

$$\mu = \frac{1}{n}\sum_{i=1}^{n} EDP_i \tag{4.1}$$

$$\beta = \sqrt{\frac{\sum_{i=1}^{n}[\ln(EDP_i) - \ln(\mu)]}{n - d_f}} \tag{4.2}$$

Here, $\phi(x)$ follows the standard Gaussian distribution.

A line (Red line in Figure 4.10 and 4.11) joining the medians of the EDPs at each FSM level is called the median FSA curve. The dispersion at every FSM level is recorded. This process is classified as Fire Stripe Analysis.

Both IFA and FSA involve scaling of fire profiles. However, FSA is a more engineered and advanced version of IFA estimating structural response as accurately as possible. IFA and FSA are time consuming methods since they require calculation of the median FSA and dispersion at each level. Therefore, there is a need to have another method which not only performs analysis quickly but also produces reasonably accurate results. Therefore, Cloud Analysis has been introduced. It is a well-established methodology in PBEE to model the relationship between EDP and IM (Bazzurro *et al.* 1998; Jalayer, 2003; Luco and Cornell, 2007). Since all these methods (IFA, FSA and Cloud analysis) use different approaches to produce the desired conditional probability of EDP for a given IM, therefore, the computation of the probability of exceedance of a given level of structural response will be different for each method.

4.4 Cloud analysis.

The method proposed here is used unconditionally, without modification from PBEE, in PSFE. It is a cost-effective method commonly used to interpret the strength of the EDP-IM relationship. Cloud analysis performs analysis for unscaled recorded intensities. Unlike FSA, the fire profiles are not grouped into bins and are not scaled to FSM levels but are rather considered as individual scenarios which are used directly in the analysis to produce the structure's response. A cloud of structural responses is generated when a structure is subjected to a suite of unscaled fire profiles, each one therefore has a different FSM value. Regression analysis is then performed to determine the EDP-FSM probabilistic relationship. Similar to

FSA, the median and standard deviation are evaluated from the regression. Traditionally, such approaches assume a linear relationship between the mean of the lognormal structural response, ln(EDP), and lognormal intensity measure, ln(FSM) as shown in Equation 4.3.

$$ln(EDP) = a + b \, . \, ln(FSM) \tag{4.3}$$

where *a* and *b* (regression coefficients) and can be determined from linear least-squares regression technique. The main steps involved in cloud analysis are as follows:

- Generate a suit of fire profiles which represent a wide range of appropriate FSMs. The random fire records are produced with the help of Monte Carlo simulation, which uses the distribution of fire development factors such as fuel load and ventilation.

- Select a type of structure to be investigated. EDPs are to be identified and measured during the structural fire analysis to assess the performance of a structure under a given fire scenario. For example, maximum displacement and maximum member temperature.

- Create a finite element model which represents the type of a structure selected and the properties of fire scenarios are entered as an input to generate the time-temperature profile to which the structure is exposed to. Perform fire analysis of the structure subjected to the generated fire records. EDPs should be monitored throughout the analysis.

- Document the peak values of the EDPs and plot them against the values of fire severity measure (FSM). Then regression analysis is performed on this cloud of data to develop a relationship between severity measures and structural engineering demand parameters. The statistical parameters of the analysis corresponding to the lognormal distribution of EDP/FSM are extracted here as the median 'μ' and standard deviation 'β' (Equation 4.1 and 4.2).

4.5 General comparison of analysis methods

Both IFA and FSA involve scaling of fire profiles. However, FSA is a more engineered and advanced version of IFA estimating structural response as accurately as possible. IFA and FSA are time consuming methods since they require calculation of the median FSA and dispersion at each level. Cloud analysis is comparatively quicker since it analyses data for unscaled fire profiles. However, in post-processing of results cloud analysis assumes a linear relationship between ln(EDP) and ln(FSM) with constant variance. As such, this assumption may be suitable only over a limited range of FSM levels. However, the benefit of this approach is that it certainly reduces the computational time, and that the distribution does consider the entire suite of results at the same time.

An example discussed in Section 4.3 to demonstrate the working of FSA is used here to present the process of Cloud Analysis. For the same suite of fire profiles (Figure 4.12), cloud analysis was also implemented. Here, structural analysis of the composite beam was performed for unscaled fire. Similar to the approach for FSA and using maximum fire temperature (MFT) as the FSM and maximum displacement as the EDP, the Cloud Analysis plot for MFT-maximum displacement is shown in Figure 4.12. Dispersion for MFT-maximum displacement relationships from the Cloud Analysis and FSA was calculated. It was observed that Cloud Analysis produced dispersion of maximum displacement as 0.169 and dispersion from FSA was 0.249 and 0.556 using "equal interval" and "equal data points" approach respectively.

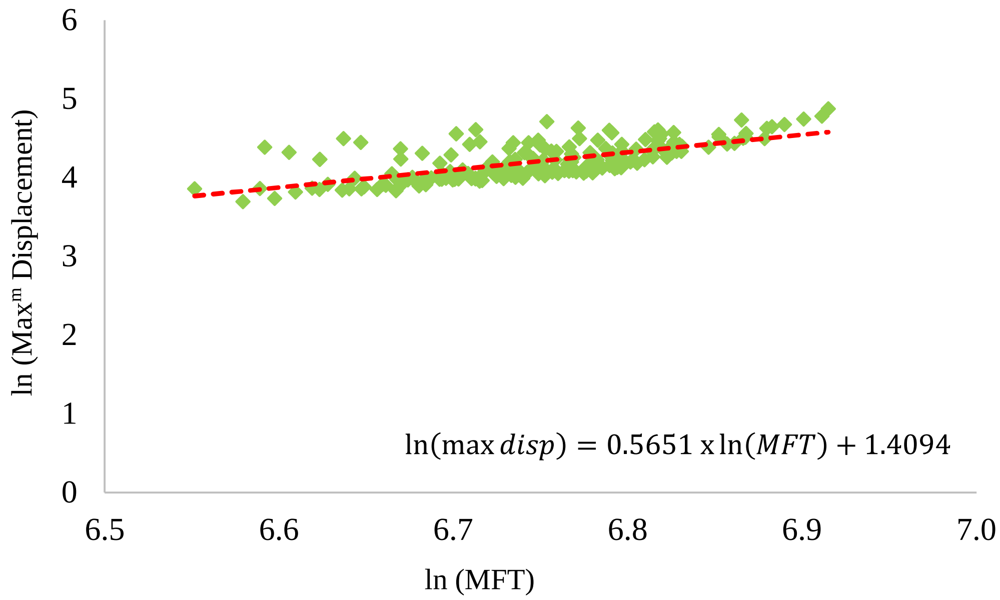


Figure 4.12: Cloud Analysis plot Maximum Fire temperature - Maximum displacement.

4.6 Summary

This chapter has presented two new analysis methods for the PSFE framework. The application of these methods was demonstrated with the help of an example. The working process and the step by step guide to implement these methods in PSFE was shown. The efficiencies of the analysis methods are investigated in more detail in the next chapter when they are implemented through a similar case study with a wide variety of FSMs. The suitability of the various FSMs and implementation of their selection criteria along with the calculation of annual rate of exceedance of fire severity and structural response for an office building in Christchurch and New Zealand is demonstrated with the extension of the example used in this chapter.

5 EFFICIENT AND SUFFICIENT FIRE SEVERITY MEASURE AND ANNUAL PROBABILITY OF STRUCTURAL RESPONSE

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2016. Efficiency of different intensity measures for probabilistic fire engineering. In: *Australasian Conference on Mechanics of Structures and Materials* XXIV (ACMSM24). CRC Press, 2016. Pp 707-712.

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2017. Analysis methods in probabilistic structural fire engineering. In: *Proceedings of the 2nd International conference on structural safety under fire and blast loading (CONFAB 2017)*: 278-278

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2018. Severity Measures and Stripe Analysis for Probabilistic Structural Fire Engineering, *Fire Technology, p. 1–27*

Shrivastava M., Abu A. K., Dhakal R. P., Moss P. J., and Yeow T. Z., 2018. “Probabilistic structural fire design using incremental fire analysis and cloud analysis”, *Proceedings of the Institution of Civil Engineers - Engineering and Computational Mechanics,* Volume 173, Issue 1, pp 11–29.

5.1 Introduction

Several FSMs, EDPs and analyses methods were discussed in Chapter 3. New analyses methods have been proposed in Chapter 4 with the help of an example. As observed in Chapter 3, the precise quantification of fire severity measures is necessary to accurately predict structural response. Therefore, selection criteria are required to be implemented to identify suitable FSMs. In this Chapter efficiency and sufficiency criteria were employed with the help of an example, following Chapter 4. A road map to implement other selection criteria and analysis method into PSFE was laid in Chapter 3. There are many more analysis methods available in PBEE which can be introduced in PSFE. The focus of this study was to demonstrate the validity of the probabilistic design process. As such the implementation of the other analysis methods was not considered paramount. The calculation of the annual rate of exceedance of fire severity is also performed in this chapter. This information is used to calculate the structural response for an office building in Christchurch and New Zealand.

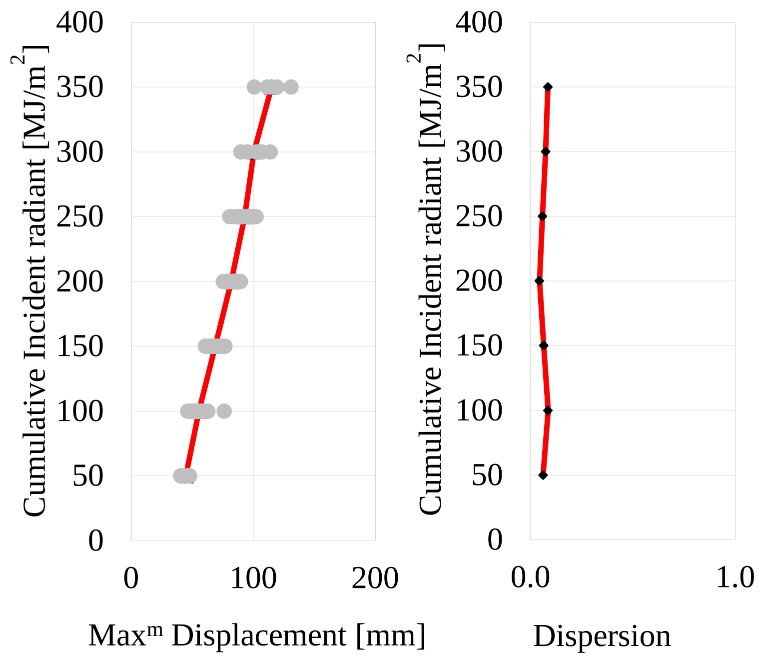
5.2 Methodology

Several fires generated in Chapter 4 with the help of distributions in fuel load and ventilation is used. In order to find out which FSM is the most efficient and sufficient each fire profile was defined by six different types of FSMs; (i) Maximum Fire Temperature (MFT), (ii) Time to Maximum Fire Temperature (TMFT), (iii) Fire Duration (FD), (iv) Area Under Curve (AUC), (v) Cumulative Incident radiation (CIR), and (vi) Fuel Load (q_{fd}). From the range of FSM values, a cumulative distribution function of the FSMs was evaluated considering a lognormal distribution. These fire profiles were used in the analysis to generate EDPs. To establish the relationship between EDPs and FSMs, Fire Stripe Analysis (FSA) and Cloud Analysis (CA) were used. The process established in Chapter 4 in implementing FSA and CA is repeated here for various FSM-EDP combinations. After establishing the FSM-EDP relationship, the selection of a suitable FSM was performed by the implementation of the selection criteria (efficiency and sufficiency). Once a suitable FSM was identified, the probability of exceedance of a given level of severity of a suitable FSM was then evaluated. Both protected and unprotected beam cases were investigated for the effect of fire protection on the selection of the efficient FSM.

5.2.1 Fire Stripe Analysis (FSA)

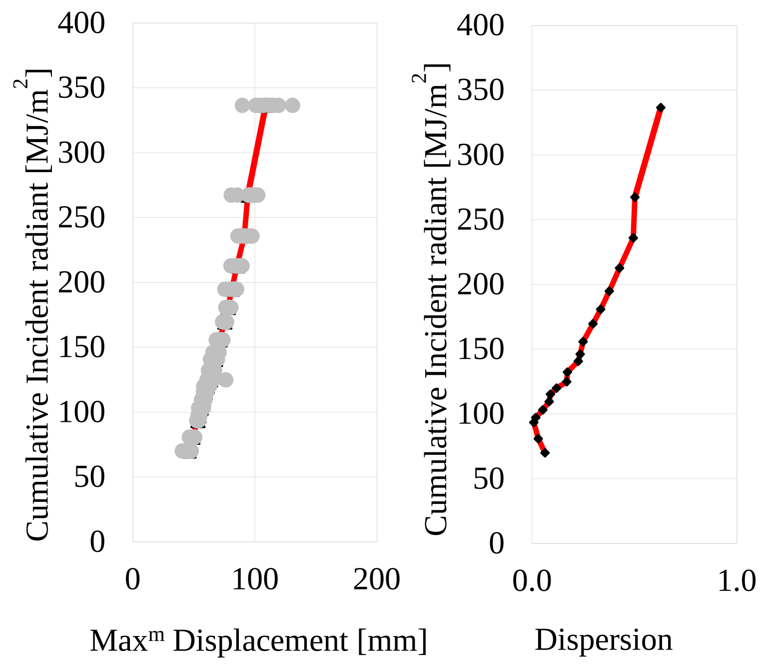
Each fire profile produced a value of various FSMs (i.e. MFT, FD, TMFT etc.). Fire Stripe Analysis was performed for each FSM-EDP combination and compared to identify efficient and sufficient FSM.

Using cumulative incident radiation as the FSM and maximum displacement as the EDP, fire profiles were grouped using two different approaches, “equal interval” and “equal data points” (as explained in Section 4.3). In the equal interval approach, FSM values (cumulative incident radiation) were grouped in bands and approximated to the nearest FSM levels - 100 MJ/m^2, 150 MJ/m^2 and so on (see Figure 5.1). In the “equal data points” approach, the cumulative incident radiations were scaled in such a way that each level has an equal number of data points (see Figure 5.2). These slightly scaled fire profiles were used as input for the thermal analysis of the composite beam exposed on three sides. Subsequent to the thermal analysis, structural analysis was performed and maximum values of vertical displacement from each analysis was recorded as EDP. The process was repeated with each fire profile, noting maximum displacement values. The maximum displacement values were then plotted for each cumulative incident radiation value. Since many fire profiles were approximated within restricted bands to one cumulative incident radiation value, many maximum displacement values get collected at each level. The median and dispersion were calculated at each level of cumulative incident radiation. Dispersion of each level was recorded separately in the dispersion curve and the



(a) (b)

Figure 5.1: FSA curve (left) and dispersion curve (right) showing the cumulative incident radiation - Maximum displacement (FSMs vs EDP) relation for equal interval approach.



(a) (b)

Figure 5.2: FSA curve (left) and dispersion curve (right) showing the cumulative incident radiation - Maximum displacement (FSMs vs EDP) relation for equal data points approach.

maximum dispersion of cumulative incident radiation and maximum displacement combination was recorded. The process was repeated to calculate dispersion for each FSM-EDP combination for both “equal data points” and “equal interval” approaches.

5.2.2 Cloud analysis

On analysing composite beam for unscaled fires, various FSM-EDP relationship, similar to MFT-max displacement relationship using cloud analysis in Chapter 4, are established (See Figure 5.3). The dispersions of each FSM-EDP were recorded for each set.

5.3 Results and Discussions

Efficiency of 6 FSMs for 2 EDPs with 2 FSA approaches and Cloud analysis method, was evaluated. Table 5.1 presents the collective information of maximum dispersion of all 6 FSMs for 2 EDPs from all the methods.

Efficiency

The comparison of the two techniques (equal intervals vs equal data points) for the FSA method produces a huge difference between the efficiency values, i.e. dispersion of FSM-EDP combination, as shown in Table 5.1. The "Equal data points" approach had a very small gap between FSM levels at a low FSM values but has a big gap at higher FSM values. This is

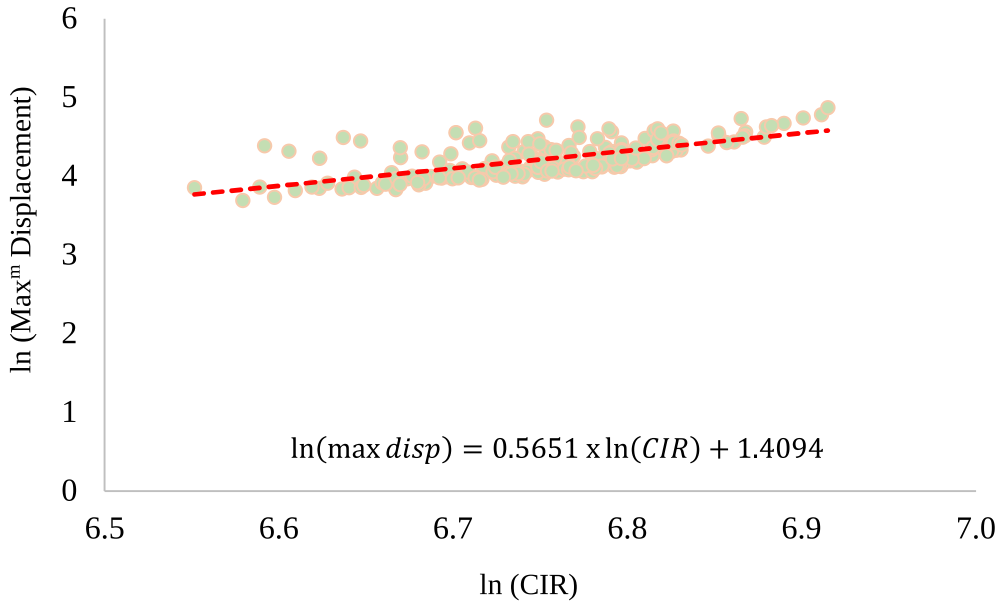


Figure 5.3: Cloud analysis curve showing the CIR - Maximum displacement (FSMs vs EDP) relation.

emphasized by examining Figures 5.1 vs 5.2 where the equal data point approach produced high dispersion as there were few data points at higher values of FSM. Thus to achieve equal data points at each level it resulted in a large interval which produces the observed large dispersion. Therefore it can be concluded that the "equal data points" approach estimated structural response poorly and was not considered further in comparison with cloud analysis.

It was also observed that though both techniques had a different approach to select FSM levels, the trend of the median FSA curve was found to be similar. To simplify the comparison of Cloud analysis and FSA, whenever a comparison was drawn with Cloud analysis, FSA with "equal interval" approach was used.

Table 5.1: Comparison of efficiency, dispersion of FSM-EDP combination, of FSMs from FSA – "Equal Data points" and "Equal Interval" techniques

	EDP = Maximum Displacement		EDP = Maximum steel temperature	
	Equal interval	Equal data points	Equal interval	Equal data points
MFT	0.249	0.556	0.232	0.353
TMFT	0.161	0.637	0.063	0.51
FD	0.169	0. 534	0.106	0.414
AUC	0.188	0.534	0.089	0.433
CIR	0.117	0.629	0.094	0.452
q_{fd}	0.121	0.604	0.101	0.398

The FSA curves for 3 FSM's, maximum fire temperature, fire duration and cumulative incident radiation with 2 EDPs (maximum displacement and maximum steel temperature) is shown in Figure 5.4 and Figure 5.5. Figure 5.4 demonstrates FSA curve (left) and dispersion curve (right) showing the FSM-EDP (maximum displacement) relation with dispersion at each FSM level

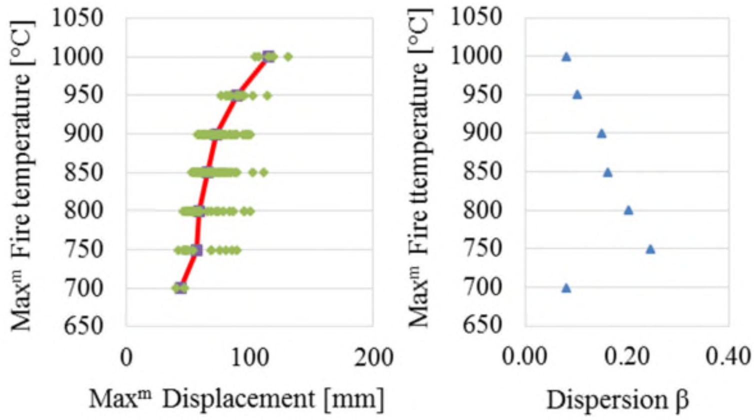


Fig. 5.4a Maximum fire temperature - Maximum displacement FSA curve (left) and dispersion curve (right).

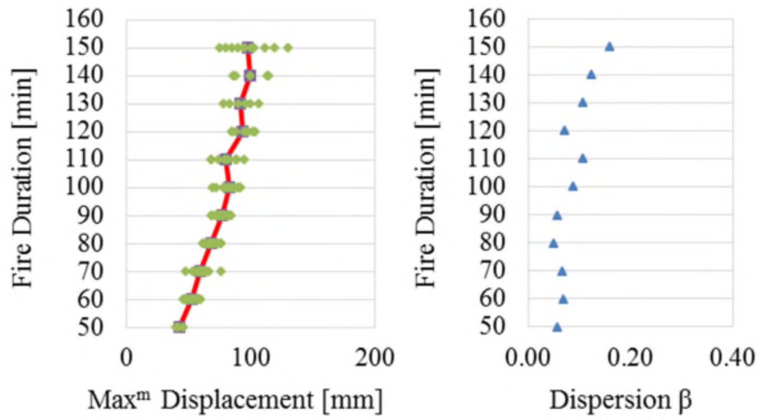


Fig. 5.4b Fire duration - Maximum displacement FSA curve (left) and dispersion curve (right).

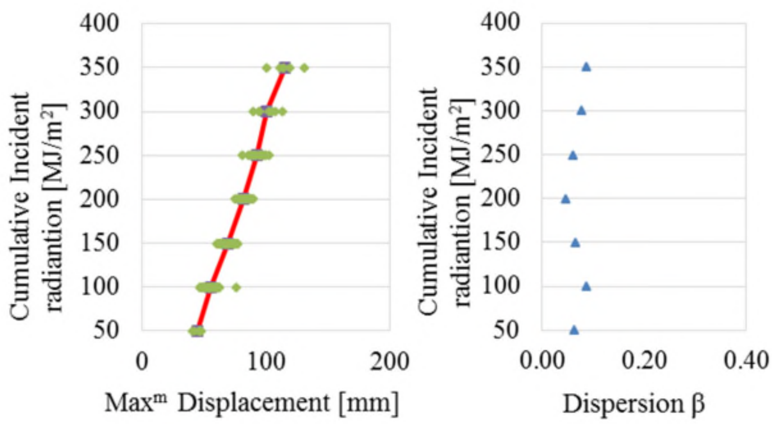


Fig. 5.4c Cumulative incident radiation - Maximum displacement FSA curve (left) and dispersion curve (right).

Figure 5.4: FSA curve (left) and dispersion curve (right) showing the FSM-EDP (maximum displacement) relation with dispersion at each FSM level to compare the efficiency of FSMs.

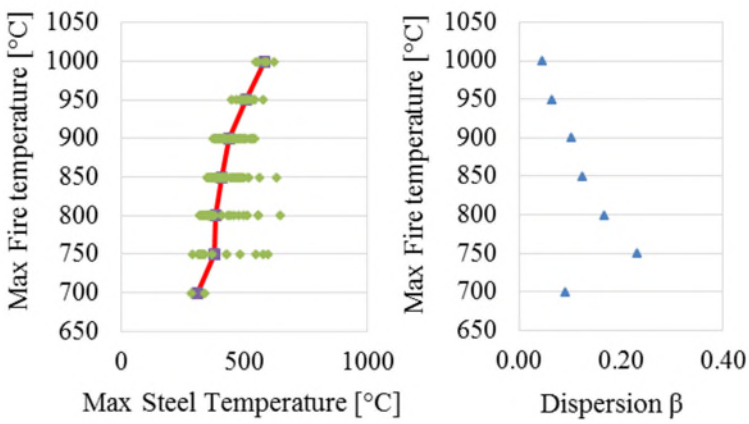


Fig. 5.5a Maximum fire temperature - Maximum steel temperature FSA curve (left) and dispersion curve (right).

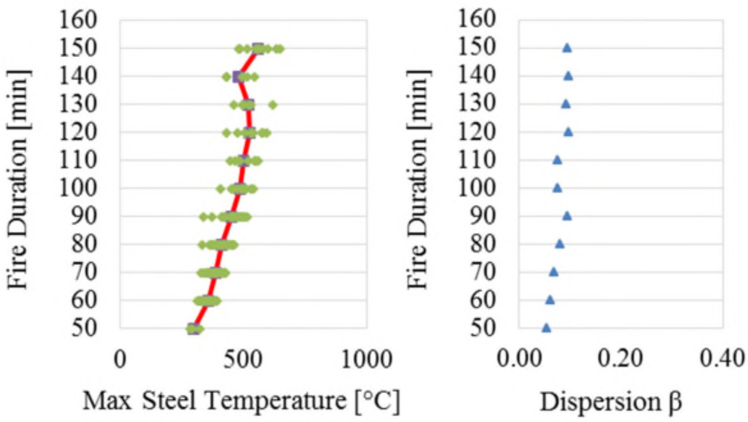


Fig. 5.5b Fire duration - Maximum steel temperature FSA curve (left) and dispersion curve (right).

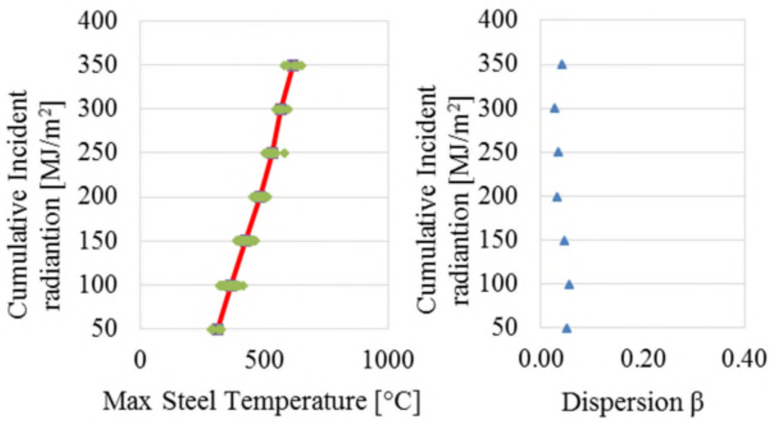


Fig. 5.5c Cumulative incident radiation - Maximum steel temperature FSA curve (left) and dispersion curve (right).

Figure 5.5: FSA curve (left) and dispersion curve (right) showing the FSM-EDP relation with Maximum steel temperature at each FSM level to compare the efficiency of FSMs.

to compare the efficiency of FSMs for maximum displacement. Similarly, Figure 5.5 demonstrates FSA curve (left) and dispersion curve (right) showing the FSM-EDP (maximum steel temperature) relation with dispersion at each FSM level to compare the efficiency of FSMs for maximum steel temperature. The efficiency of all the FSM candidates were recorded and summarized in Table 5.2 and Figure 5.6 and 5.7, where their maximum dispersions are compared for both Fire Stripe Analysis (FSA) and Cloud Analysis (CA). In FSA, the results of the analyses show that for a protected composite beam CIR is the most efficient FSM for both maximum vertical displacement and maximum steel temperature. This study was also performed on an unprotected composite beam, and for that q_{fd} is found to be the efficient FSM for maximum vertical displacement and MFT was found to be an efficient FSM for maximum steel temperature (MST), see Figure 5.8 and 5.9. It is appropriate to find MFT as an efficient FSM for MST for an unprotected beam because the member temperature profile closely follows the fire temperature. Figure 5.6 and Figure 5.7 demonstrates that efficient FSMs produce considerably lower dispersion for the EDP than the other potential FSMs. In Cloud Analysis, the results of the analyses show that for a protected composite beam CIR is the most efficient FSM for both maximum vertical displacement and maximum steel temperature, see Figure 5.6 and 5.7.

Table 5.2: Comparison of efficiencies, i.e. dispersion of FSM-EDP combination, of FSMs from FSA and Cloud analysis

	EDP = Maximum Displacement		EDP = Maximum steel temperature	
	FSA	Cloud Analysis	FSA	Cloud Analysis
MFT	0.249	0.169	0.232	0.098
TMFT	0.161	0.112	0.063	0.063
FD	0.169	0.090	0.106	0.081
AUC	0.188	0.084	0.089	0.041
CIR	0.117	0.036	0.094	0.017
q_{fd}	0.121	0.100	0.101	0.084

Figure 5.6 and 5.7 compared FSM efficiency to estimate maximum displacement and maximum steel temperature (MST) respectively from both methods (FSA and Cloud analysis). It is evident that FSA produces higher dispersions because Cloud analysis (CA) assumes a linear relationship between EDP and FSM with constant variance, whereas, in FSA, fire profiles were scaled within the band to the corresponding FSM level and dispersion was calculated at that FSM level. This bin approach in FSA gathers more data points at a given

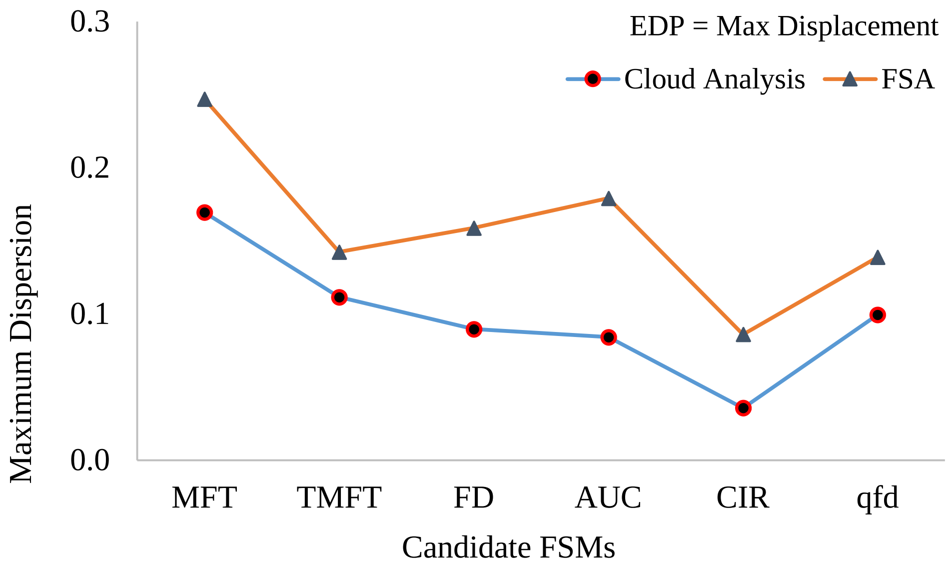


Figure 5.6: Efficiency comparison of FSMs for maximum displacement.

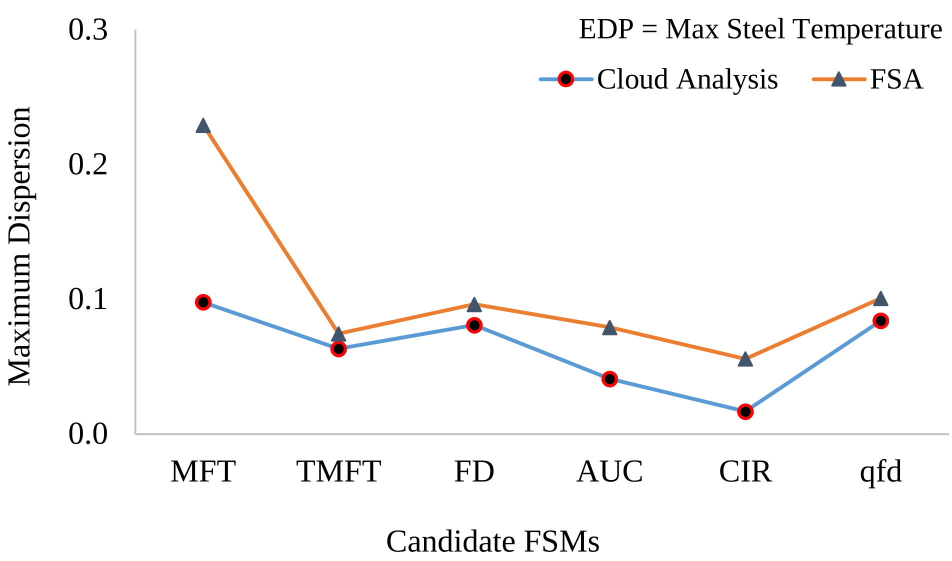


Figure 5.7: Efficiency comparison of FSMs for maximum steel temperature.

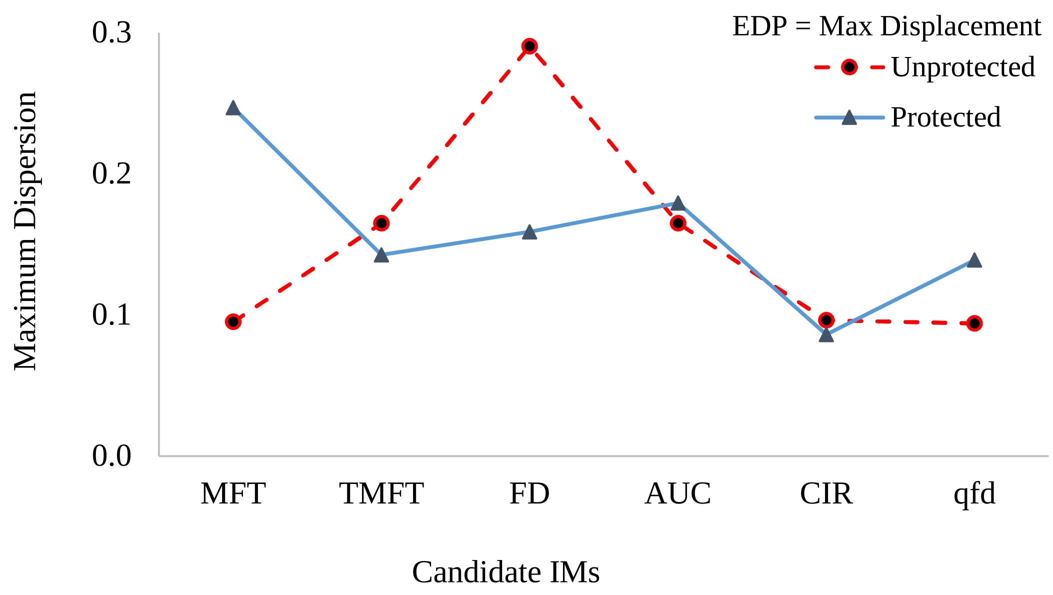


Figure 5.8: Efficiency comparison of FSMs for maximum displacement

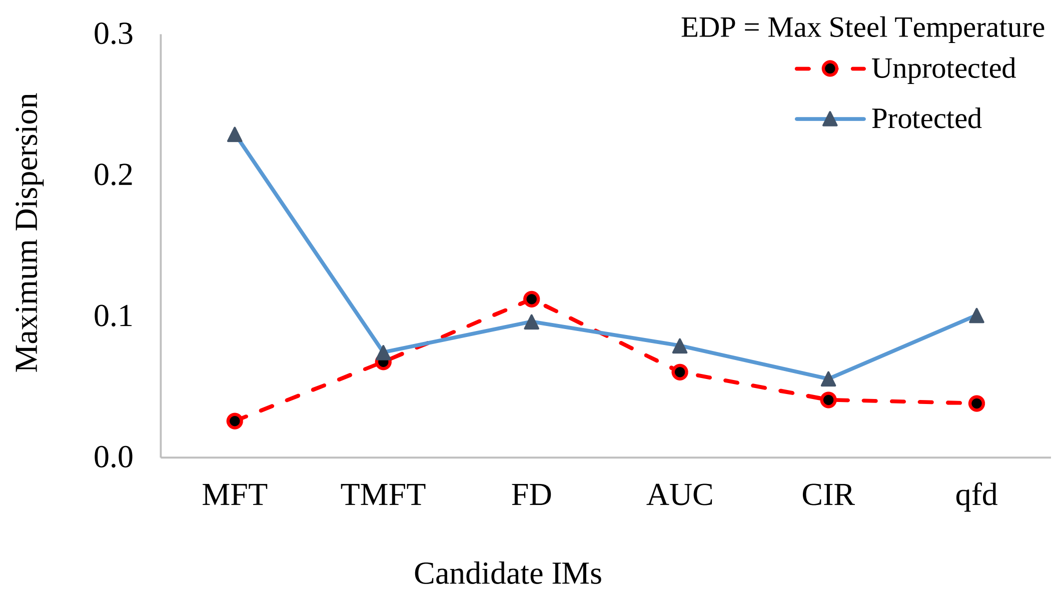


Figure 5.9: Efficiency comparison of FSMs for maximum steel temperature.

FSM level which increases the dispersion, but at rare events it has fewer data points. The number of observations used in the calculation of dispersion in FSA is less in comparison to CA. This difference definitely affects the efficiency of FSMs.

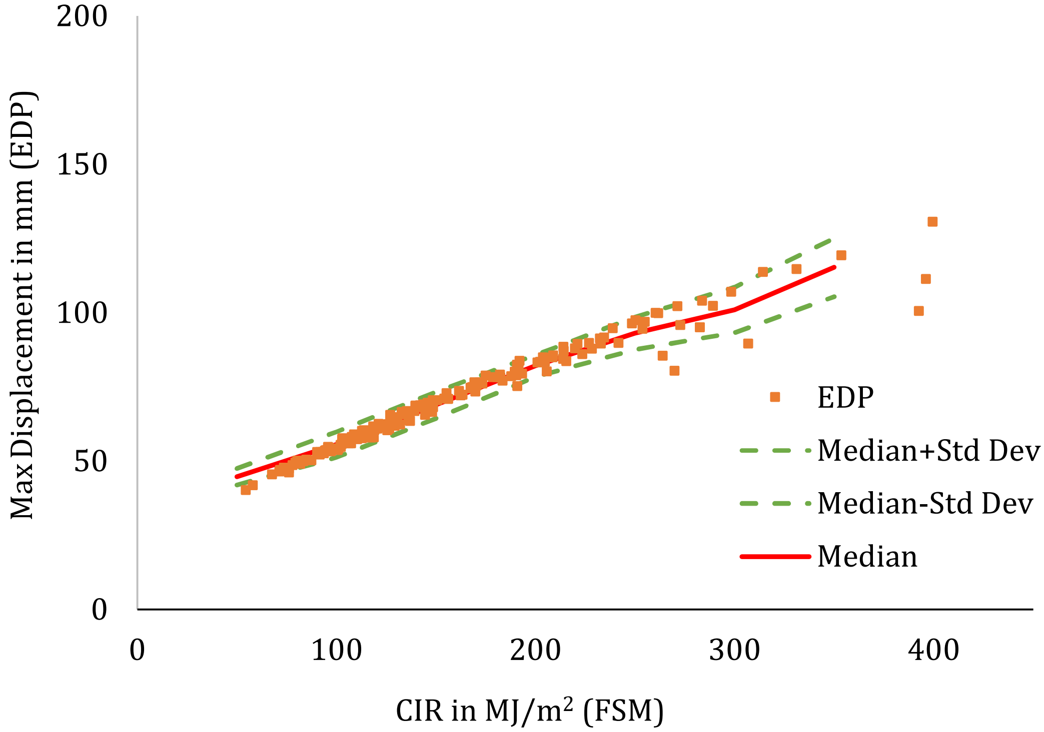


Figure 5.10: Median FSA curve with raw EDP data points

It should be noted that both analysis methods show CIR as an efficient FSM (minimum dispersion) in estimating maximum displacement, whereas in estimating maximum steel temperature FSA results in TMFT being the efficient FSM, while Cloud analysis results in showing CIR efficient, although there is a big difference in the dispersion of EDP for given CIR from both methods, see Table 5.1 and 5.2. This reveals that although there are differences in the outcome, the conclusion (i.e. efficient intensity measure) of the analyses is similar for maximum displacement.

Figure 5.10 shows the median FSA (equal interval approach) with ± standard deviation along with the raw data points. The median FSA adequately represents the raw data. Figure 5.11 illustrates the median EDPs, according to Cloud analysis with ± standard deviation along with the raw data from the thermal and structural analysis. The results may be compared in other ways, such as comparison of medians and standard deviation as illustrated in Figure 5.12 and 5.13 respectively. The mean of ln(EDP|FSM) was estimated and compared as shown in Figure 5.12 for the CIR-maximum displacement combination. Similarly, the estimated standard deviations of EDPs can be compared with FSA and Cloud analysis in Figure 5.13. The cloud analysis has a constant standard deviation of EDP, whereas FSA estimates a standard deviation

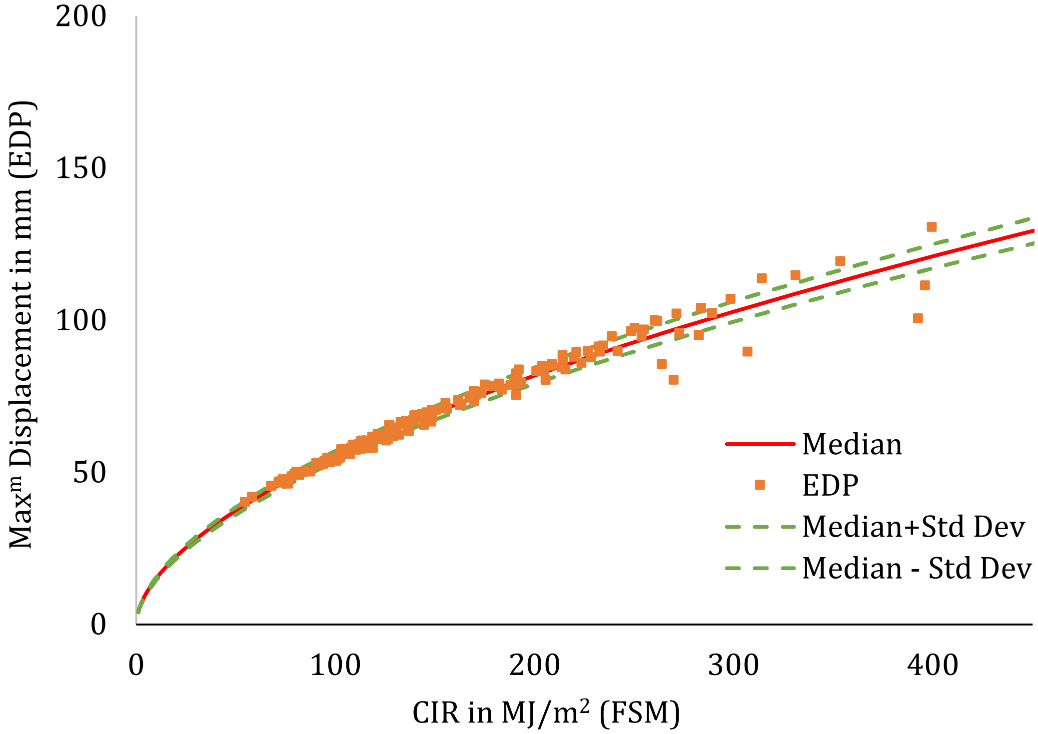


Figure 5.11: Median Cloud analysis curve with raw EDP data points

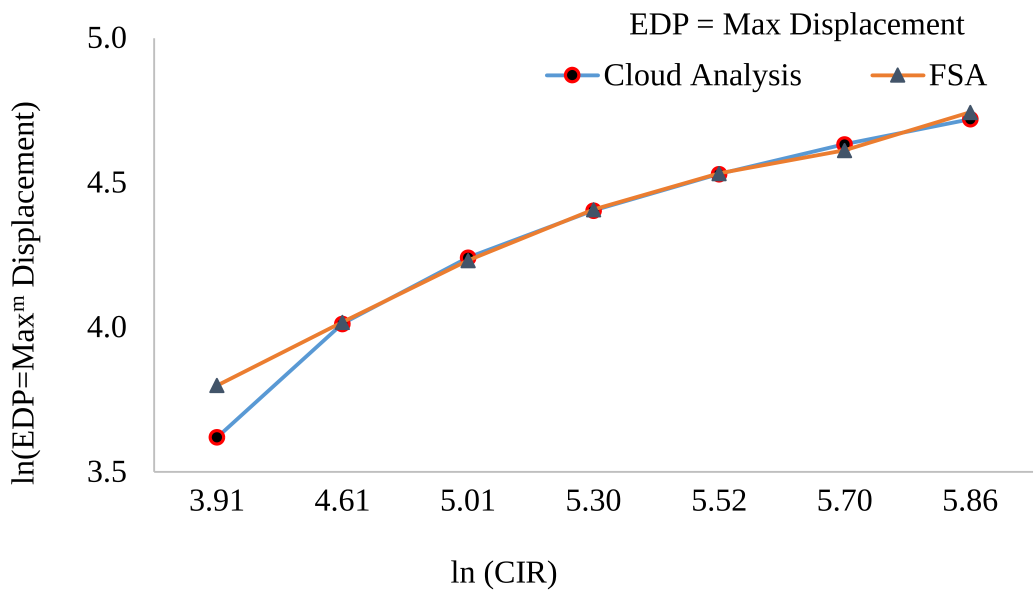


Figure 5.12: Comparison of median of CIR-maximum displacement from FSA and CA.

at each FSM level and assumes a linear fit between the levels. For other FSM-EDP combinations, similar comparisons may be drawn. It has been observed from Figure 5.12 that

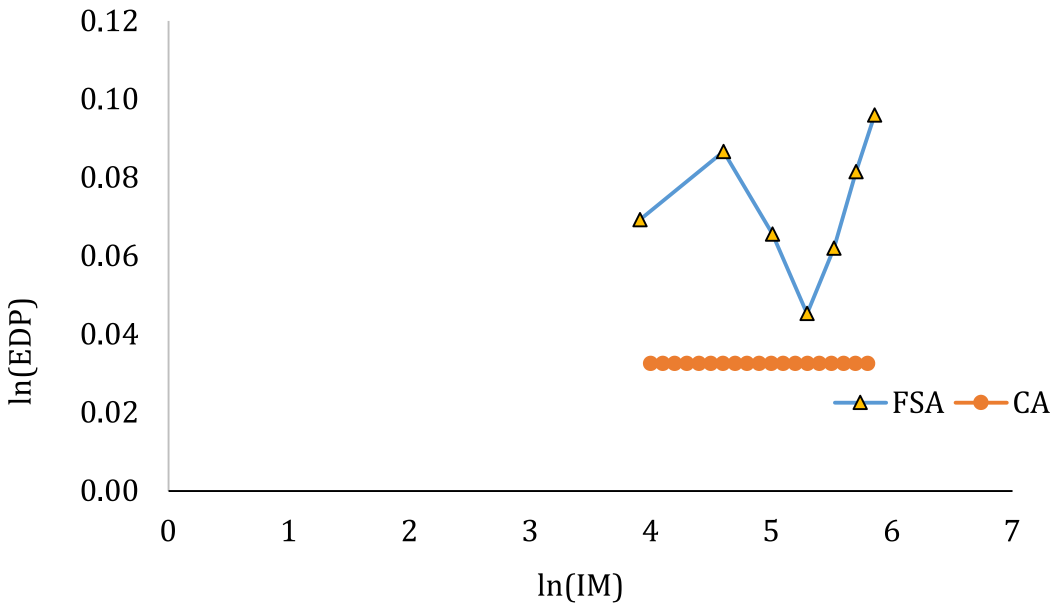


Figure 5.13: Comparison of Standard deviation of CIR-maximum displacement from FSA and CA.

both analysis methods produce very close median curves which reflect the suitability of either method for establishing a relationship between FSM and EDP.

Sufficiency

A sufficient FSM is when the structural response for a given hazard is independent of the variations in the key elements of the fire hazard. It signifies that the selection of records is not significant. The structural response will be estimated precisely even there is a variation in key elements of the fire hazard of the selected fire records. The major key elements of fire hazard are fuel load and ventilation. Therefore, an FSM should be sufficient with respect to fuel load and opening factor for a finite number of fire profiles. A sufficient FSM should not distinguish between a short duration high-temperature fire and a long duration low-temperature fire, if they produced the same structural response. The sufficiency of an FSM was evaluated by the extent to which the residuals of EDP (the difference in the predicted value and the actual value of EDP) shows no trend in the correlation with fuel load and opening factor. Sufficiency of FSMs was evaluated using the FSA method. Here residuals are obtained from the FSA curve, as the difference of mean FSA value at any FSM level to the actual EDP value at that level. Regression analysis was performed between the obtained residuals of EDP and the corresponding fuel load or opening factor. No observed trend in the regression line of the residuals with respect to fire development parameters (fuel load and opening factor) indicates the sufficiency of an FSM. Numerically, sufficiency is quantified by determining the p-value,

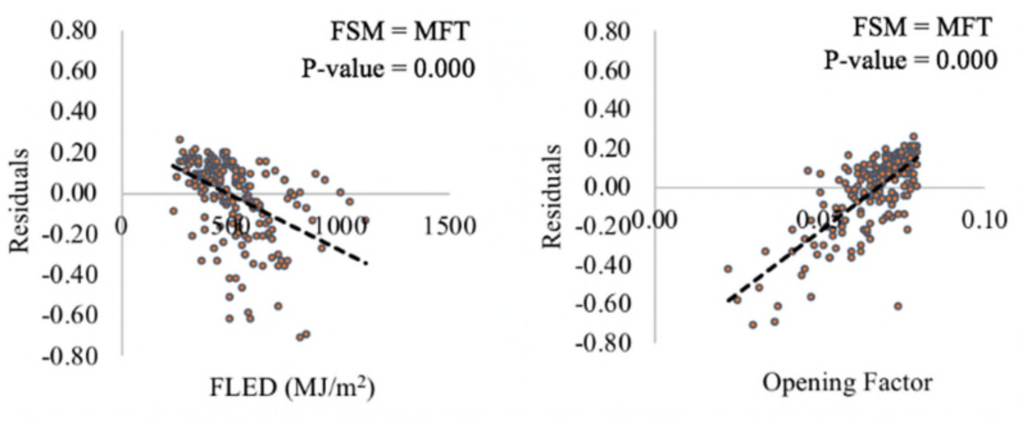


Figure 5.14: MFT sufficiency with respect to (i) Fuel load and (ii) Opening Factor in predicting maximum displacement of protected composite beam

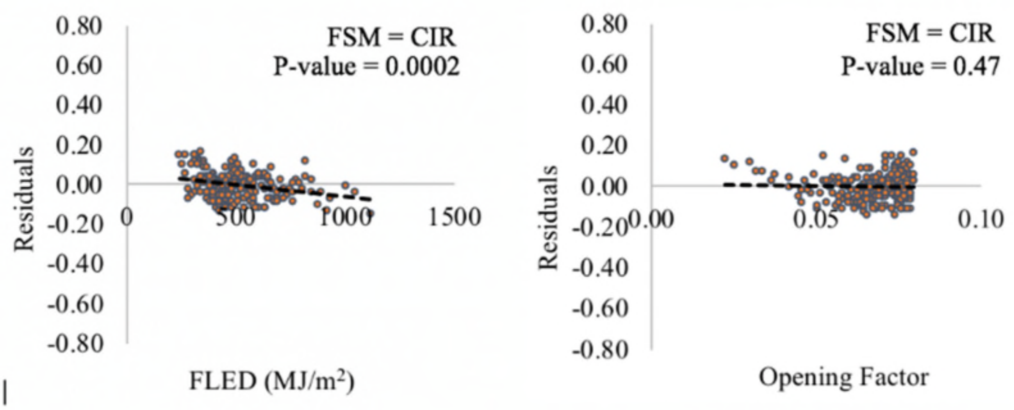


Figure 5.15: CIR sufficiency with respect to (i) Fuel load and (ii) Opening Factor in predicting maximum displacement of protected composite beam

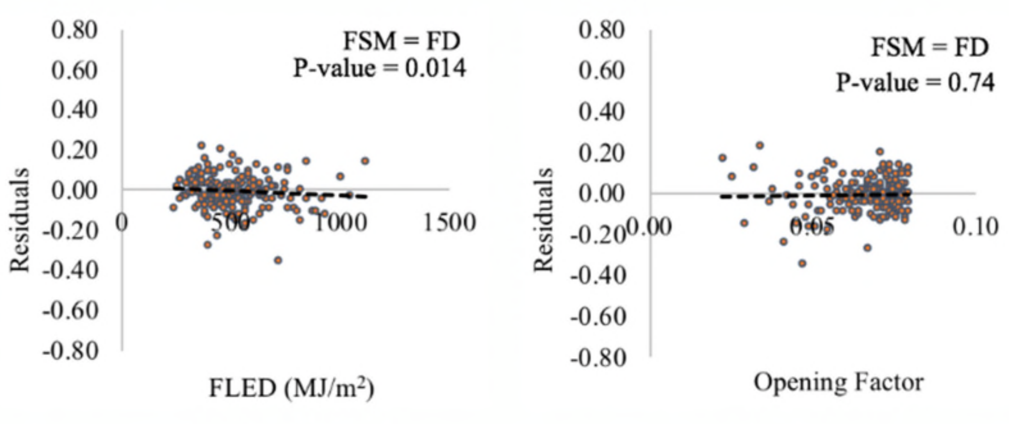


Figure 5.16: FD sufficiency with respect to (i) Fuel load and (ii) Opening Factor in predicting maximum displacement of protected composite beam.

which is defined as the probability that the slope of the regression line is equal to zero. If the p-value is less than 0.05, it provides enough evidence to reject the null hypothesis i.e. that the slope of the regression line is zero.

For illustration, the regression of residuals of maximum displacement of the protected composite beam for maximum fire temperature (MFT), cumulative incident radiation (CIR) and Fire Duration (FD) with respect to fuel load energy density (FLED) and opening factor (OF) are shown in Figure 5.14, 5.15 and 5.16 and recorded in Table 5.3. The remaining graphs are shown in Appendix. It can be seen from the figures that for CIR and FD, the p-value is greater than 0.05. Although all FSMs produce p-values less than 0.05 with respect to FLED, FD and CIR visibly show no trend in the regression line and have comparatively higher p-value than others. Similarly, for MST as an EDP, TMFT produced a p-values of 0.2 for opening factor, and CIR produced 0.13 for FLED as shown in Table 5.3.

The above calculation process indicates that CIR is the most efficient FSM, and CIR and FD are the most sufficient FSM with respect to opening factor for the evaluation of maximum displacement. CIR is also sufficient with respect to FLED for estimating MST. The prediction of efficient and sufficient FSM leads to a better estimation of maximum vertical displacement and maximum steel temperature. The identified suitable FSMs in the study are applicable to a composite structure, though results from the other research indicate that these are applicable for general structures too (Moss *et al.* 2016).

Table 5.3: Sufficiency comparison using P-value for FSMs.

	EDP = Max Displacement		EDP = MST	
	FLED	Opening Factor	FLED	Opening Factor
MFT	0.00	0.00	0.00	0.00
TMFT	0.00	0.00	0.00	0.20
FD	0.01	0.74	0.00	0.00
AUC	0.00	0.00	0.00	0.00
CIR	0.00	0.47	0.13	0.00

Equation 3.2 (in Chapter 3) implies that PSFE has the capability to provide information on the annual rate of exceedance of damage and repair cost/time incurred to the structure during its lifetime.

In order to estimate the mean annual rate of exceedance of a fire hazard of different severity (ϕ_{FSM}), the product of the probability of occurrence of the fire (i.e. annual occurrence rate of fire, r_{fi}) and the probability of exceedance of the fire hazard of different severity *P(FSM)* was required. The probability of occurrence of fires was evaluated for Christchurch and New Zealand.

5.4 Probability calculation of severe structure fire in an office building in CHCH and NZ.

The mean annual rate of exceedance (MARE), i.e. $\phi_{maxtemp}$, of the maximum temperature in a compartment is found by the product of the probability of exceedance i.e. $P(T_{max})$ and the probability of a structural fire per year (r_{fi}) as shown in Equation 5.1.

$$\emptyset_{maxtemp} = P(T_{max}).r_{fi} \tag{5.1}$$

The collection of maximum fire temperatures from the various fire profiles is used to estimate the probability of exceedance a specified value of maximum temperature in a compartment [*P(Tmax)*]. In other words, $P(T_{max})$ is the probability of any particular value to exceed from the data. This chapter evaluates MARE of the suitable FSM for Christchurch and New Zealand, which is based on a number of selection criteria, as discussed in Chapter 3.

For the purpose of this section, information was sought on the probability of occurrence of structural fires in Christchurch and New Zealand per year. Statistics of fires in New Zealand was obtained from a report “Emergency incident statistics” by the New Zealand Fire Service (NZFS, 2013). It covered a range of fires which required the intervention of the fire service to extinguish. All small fires which could self-extinguish or were extinguished by the building occupants were not included in the statistical data, as they were no threat to the structure.

The report by NZFS classifies fires “Structure fires” are classified as those that occur in buildings and cause damage from 1% to 100%, and include incidences such as flame damage, smoke damage and water damage. There is no damage threshold mentioned in the report to classify the damage level of the structure but there is some information about the percentage of property saved. Therefore, any fire which produces damage to a structure as described above, was classified under the “Structure Fire” classification. Fires which did not reach flashover and died out before affecting the structure were considered under the classification of “structure fires with no damage”. The structure may also be of any type i.e. residential or commercial. The data is shown in Figures 5.17 and 5.18 and Table 5.4. Looking at Table 5.4, the total number of fire calls recorded in 2012 were 21946 in NZ and 4524 in Christchurch, out of which the total number of structure fires (i.e. fires in structures/buildings) in New Zealand were 5434, 116 in Christchurch CBD (Central Business District) and 359 in the wider Christchurch area. The underlined values in Table 5.4 were unknown and calculated based on the population ratio as outlined below.

As mentioned previously, this research focusses on office buildings. According to the NZFS report, the number of office fires in New Zealand was 180 in 2012/13 but was unknown for Christchurch city. Therefore, the percentage of an office structure fire in New Zealand was estimated to be 3.31% (180/5434 x 100 = 3.31%) per year. Since the NZFS report does not provide the number of structure fires for various occupancies for different cities, the same percentage of office structure fire for Christchurch city as of New Zealand was assumed. This provided the number of office structure fires for Christchurch as 12 (3.31% of 359). The number of commercial premises in Christchurch city area is 1740. These offices are assumed to be linked with the full Christchurch population (i.e. 341500). Based on the above calculation the probability of office structure fire per year (r_{fi}) was given by:

$$r_{fi} = \text{No. of office structure fire in a year/Total no. of office buildings} \tag{5.2}$$

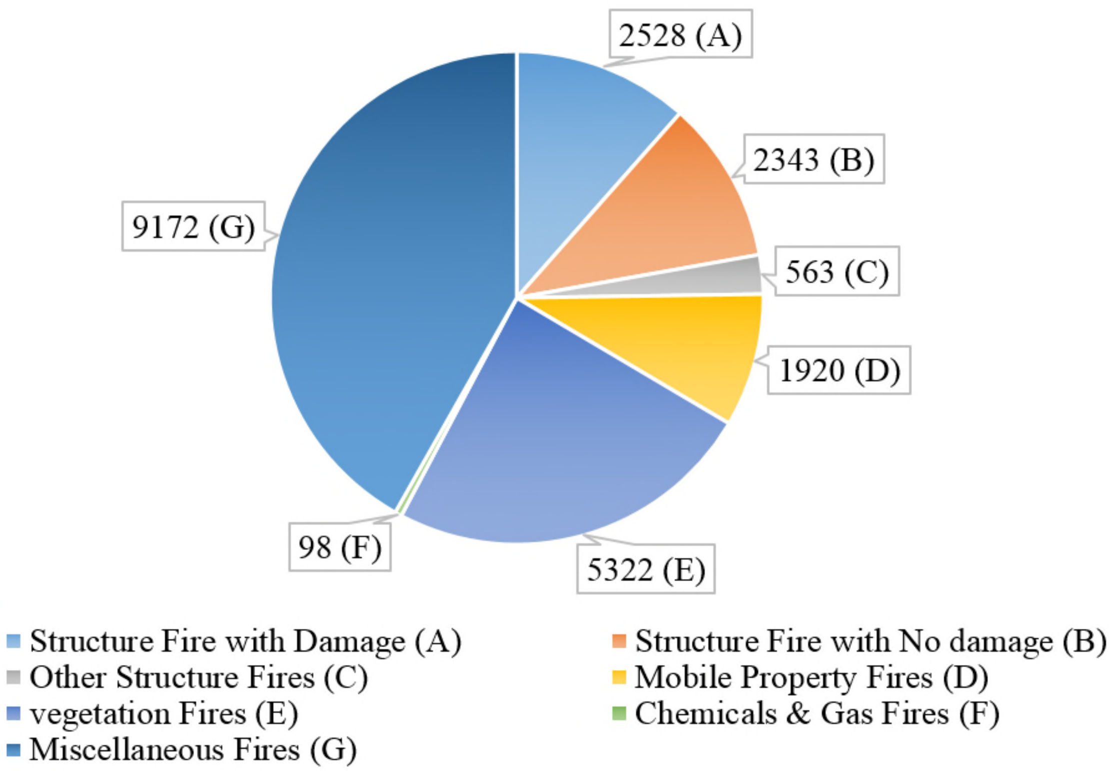


Figure 5.17: Fire Statistics of New Zealand of year 2012/13 (NZFS, 2013)

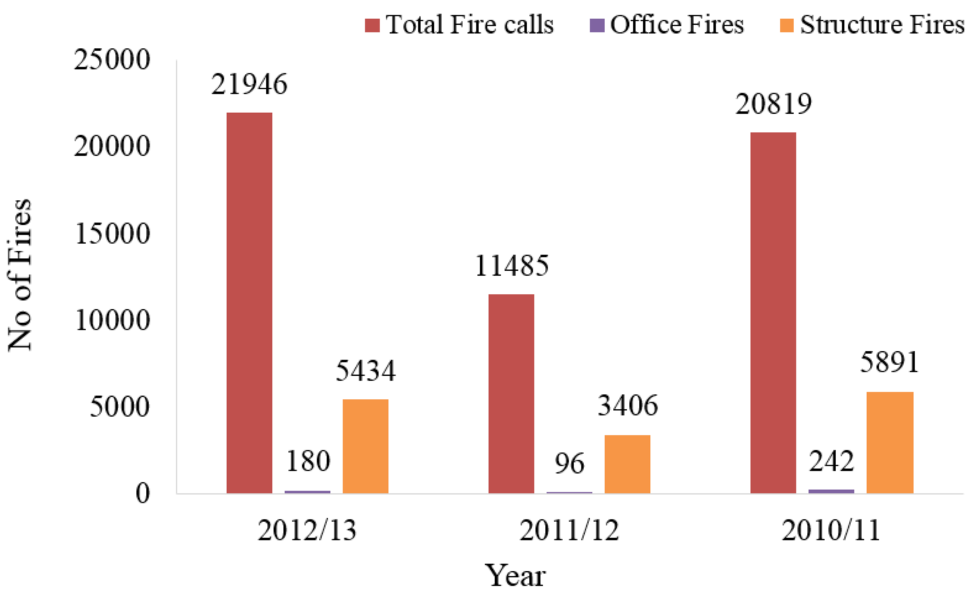


Figure 5.18: Fire Statistics of NZ for three consecutive years

The above equation (Equation 5.2) gives the r_{fi} for Christchurch as 0.007 (= 12/1740).

Table 5.4: Fire and Structure statistics of NZ and Christchurch

Factors	New Zealand	Christchurch
Population	4242000	341500
Total Fire calls	21946	4524
Structure Fire	5434	359
Office fire	180	*12*
No of office building	*21616*	1740

In order to have a similar calculation for the New Zealand as a whole, Equation 5.2 parameters for New Zealand need to be calculated. The number of office structure fires in New Zealand was 180. Since the total number of office buildings as not available for New Zealand, therefore extrapolation of numbers for an office building in Christchurch to New Zealand based on the population was performed. Christchurch had 1740 office buildings with a population of 341469. Considering the same ratio of office building per person (1740/341500 = 0.005), New Zealand was expected to have 21616 (=0.005 x 4242000 = 21616) office buildings for 4242048 people. r_{fi} for New Zealand was calculated using Equation 5.2 as 0.0083 (= 180/21616).

This indicated that Christchurch has a lower probability of occurrence of a structure fire in office buildings as compared to the entire country. This helps to calculate the annual rate of occurrence of fire and the structural response for office buildings. The above discussed probability calculation with certain assumptions could be improved with more recent information of the number of offices and the number of office fires at the location (Christchurch city or New Zealand as a whole).

5.5 Annual probability of exceedance of Fire hazard and structural response

It is observed that Christchurch has a lower probability of occurrence of a "structure fire" in office buildings as compared to the entire country. This can be used to calculate the annual rate of occurrence of fire and the structural response for office buildings.

The hazard curve (assuming a normal distribution) showing the mean annual rate of exceeding (MARE) a given value of CIR (the most suitable FSM) for both a New Zealand and a Christchurch office building was calculated with the help of Equation 5.1. CIR is the total incident radiant heat flux to which a structure is exposed. The cumulative incident radiation will only consist of the radiative contribution of the fire. It was evaluated using Equation 3.4. P(T_{max}) was evaluated based on the data of CIR collected for each fire profile and r_{fi} was calculated here for both New Zealand and Christchurch city. Using Equation 5.1, MARE of CIR was shown in Figure 5.19.

The data is very useful for design purposes since it indicates the probability of exceeding a CIR value in a fire compartment. Therefore, the designer need not perform fire analysis since this graph can be used to observe the fire scenario in a compartment of an office building. A similar interpretation can be drawn for other FSMs such as maximum fire temperature since it is a useful parameter for the response calculation of the member.

From Figure 5.19 it can be shown that the probability of the cumulative incident radiation to exceed 10 MJ/m^2 is 0.8% whereas exceeding 300 MJ/m^2 is very unlikely for both regions. The value of CIR for equivalent exposure under the standard fire for 30 and 60 minutes is 80 and 234 MJ/m^2 respectively. For comparison, the standard fire requires 8.5 minutes to reach 10 MJ/m^2 of CIR and 70.5 minutes for 300 MJ/m^2. The same interpretation can be drawn for the other FSMs based on the illustrated curves.

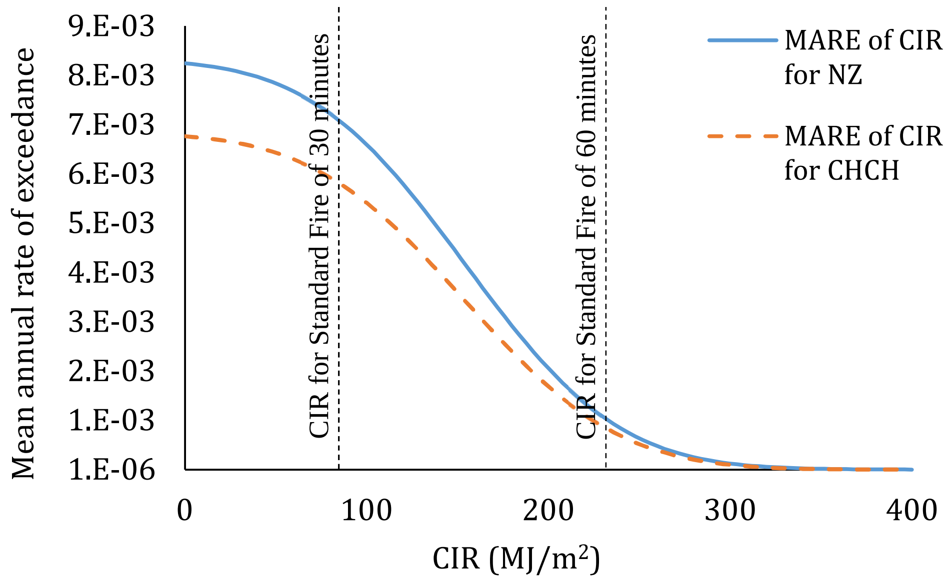


Figure 5.19: Hazard curve for CIR

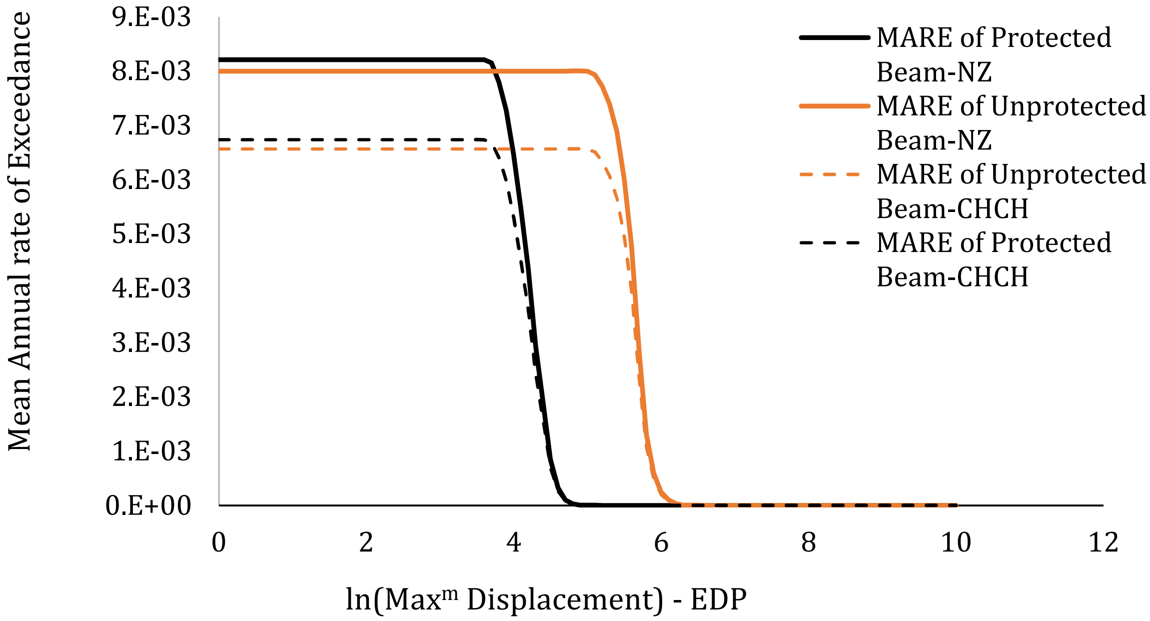


Figure 5.20: Mean Annual rate of exceeding a given level of maximum vertical displacement

Similar to the hazard curves for CIR, the exceedance curves for the EDPs are calculated. The probability density function for EDP is assumed to be a normal distribution. The EDP curve was derived by using CIR as the FSM. G(EDP|FSM) was calculated and integrated with respect to λ(FSM). Figure 5.20 shows the MARE curve for EDP (max displacement) for both protected

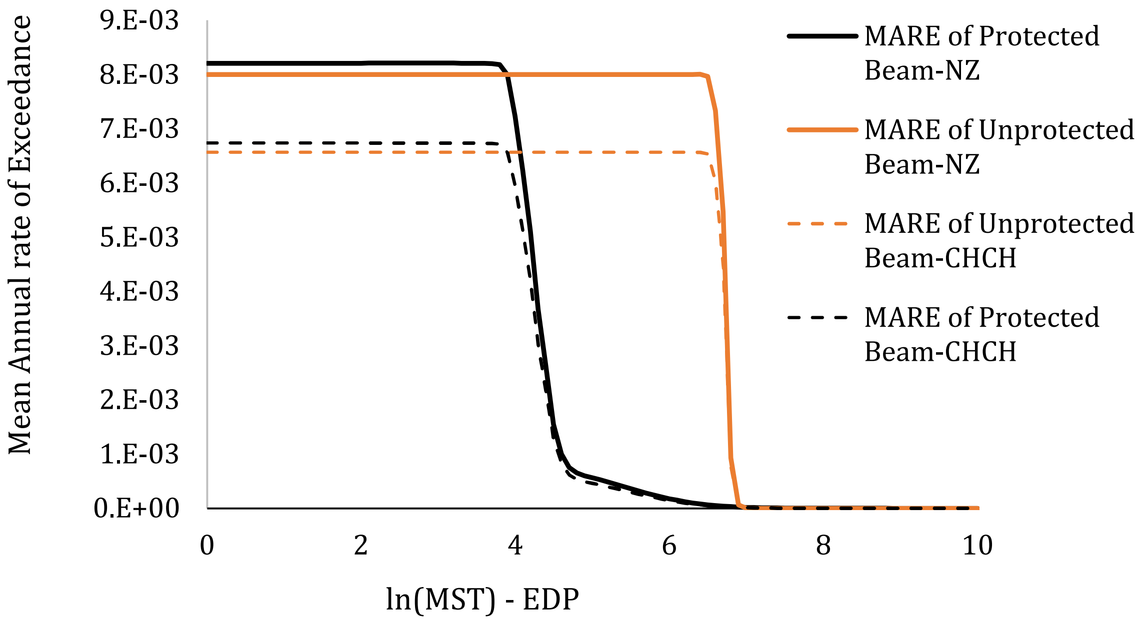


Figure 5.21: Mean Annual rate of exceeding a given level of max steel temperature

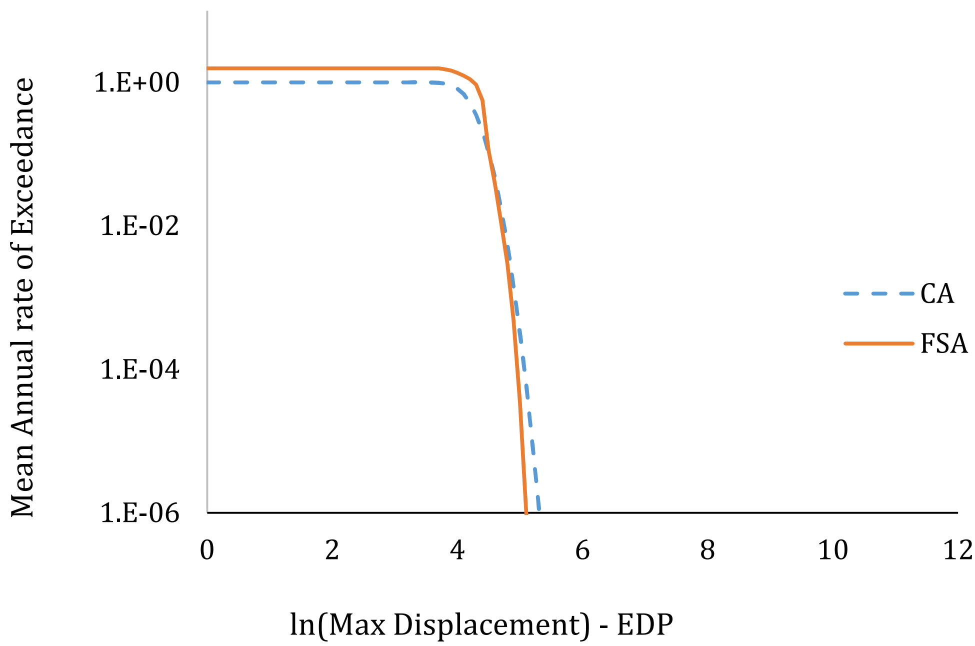


Figure 5.22: Mean Annual rate of exceeding a given level of maximum vertical displacement from FSA and Cloud analysis

and unprotected conditions. Similarly, the MARE curve is drawn for maximum steel temperature as an EDP (Figure 5.21).

It is clear from the exceedance curve for the EDPs that the protected beams reduce the probability of exceedance of larger deflections. This may be extended to calculate the annual expected loss which may be useful in planning fire protection strategies.

EDP hazard curves are compared in Figure 5.22 from both analytical methods (FSA and Cloud Analysis). It is observed that there is much less difference in the curves. For lower EDPs, FSA produces slightly higher MARE and for higher EDP, CA produces slightly higher MARE. FSA has a greater dispersion, which results in a greater probability of exceedance at lower CIR. FSA has few data points in larger events, resulting in perhaps a poorer estimate of structural response in this range. The EDP also appears to be low, which results in lower probability of EDP for rarer events.

5.6 Conclusion

A composite beam from a typical NZ office building was used to demonstrate the process of selection of an efficient and sufficient FSM through the use of two analysis methods- FSA and cloud analysis. The beam, in both protected and unprotected conditions, was exposed to a family of fires. Two techniques for the selection of FSM band (or FSM interval) were implemented in this chapter: ‘equal intervals’ and ‘equal data points’. Both approaches are applied to the same set of data, the dispersion is different due to the different nuances of each approach. It was concluded that an “equal data point” approach produces higher dispersion and therefore not recommended. FSA (using the equal interval approach) produced higher dispersions of structural response due to categorising the FSM range into bands and evaluating the mean and dispersion at each level. There were fewer data points at low or high FSMs as compared to the intermediate FSMs level which resulted in higher dispersion of response in the lower and higher levels. It was found that for the protected composite beam CIR was the most efficient FSM for both maximum vertical displacement and maximum steel temperature. CIR and FD were the most sufficient FSM with respect to opening factor for the evaluation of maximum displacement. CIR was also found to be sufficient with respect to FLED for estimating maximum steel temperature (MST). The prediction of efficient and sufficient FSM leads to better estimation of maximum vertical displacement and maximum steel temperature. The comparison of results for unprotected and protected beams revealed information about the advantage of fire protection in limiting structural failure.

The annual rate of exceedance of hazard intensity and structural response was also calculated for New Zealand and Christchurch city with the help of the probability of occurrence of a “structure fire” and the probability of exceedance of a given level of FSM given an EDP. It was observed that Christchurch city has 15% less probability of exceedance of the specified fire severity level than the whole of New Zealand. The method also compared the structural response hazard curves, i.e. the EDP hazard curves. Due to greater dispersion at lower CIR, FSA produced higher probability of exceedance of structural response for smaller EDP and because of fewer data points at higher CIR values, it resulted in lower probability. It is concluded that since cloud analysis is a direct approach which does not involve any scaling and uses raw fire hazard data for the analysis, it therefore produces more economic results in comparison to FSA. FSA is a unique approach in PSFE and will be very useful where fire profiles fall in a small FSM band. These results can be extended to calculate annual loss or damage of a structure in NZ or Christchurch. This work can be executed on framed structures (in both steel and concrete) with further extension to damage and loss estimation.

6 EFFECT OF MODELLING ON FAILURE PROBABILITIES IN STRUCTURAL FIRE DESIGN

Shrivastava M., Abu A. K., Dhakal R. P., and Moss P. J., 2018. Effect of modelling on failure probabilities in structural fire design. In: *Proceedings of the Sixth International Symposium on Life-Cycle Civil Engineering (IALCCE2018)*, Ghent, Belgium.

6.1 Introduction

Prescriptive structural fire design requires strict adherence to a set of rules that have little or no scientific justification but are practical ways that have traditionally been used to provide safety. In this approach, a building nominally rated to a given fire resistance rating (FRR) suggests that all individual structural elements in the building are rated to that same FRR. This approach gives nominal fire safety, but it does not provide any measurable sense of reliability of the structure for the multiple realistic fire scenarios which may occur in the structure. The probabilistic methodology balances this absence of quantification of reliability and proposes a technique to gauge reliability of structures. A pertinent methodology of measuring the reliability of a structure is to estimate the probability of failure. Probability of failure is not only a reliable indicator but also a valuable tool from a design point of view. The annual probability of exceedance of structural response, which can be extended to calculate failure probability, has been calculated in Chapter 5 for office buildings in Christchurch and New Zealand. An example discussed in Chapter 4 and 5 used a 2D steel-concrete composite beam element to calculate structural response for various fires. The process of obtaining structural response for fire conditions has transformed over the past few years. Nowadays it is more common to model a 3D finite element structure. This is possible mostly due to improved computational power and the availability of suitable software due to the increased research in structural fire engineering. Modelling complex 3D structures using advanced computational software produces huge results and may lead to the possibility of overlooking certain errors. Adequate knowledge of structural response and finite element modelling may aid the identification of such mistakes. The user needs to verify these results to prevent a failure of the structure. Furthermore, force and stress distribution in the structural member is influenced by the different ways of modelling. Engineers may produce different results for the same building having different approaches to modelling. However, computation time increases with the size and complexity of the model.

Structural response could be different if modelled under different configurations, such as an isolated member (2D) or as part of a 3D structural system. The behavior of a structural element should be the same irrespective of the modelling configuration. However, the difference in the response occurs due to the introduction of limitations due to different boundary conditions imposed on the structural element because of the modelling configuration. The chosen support conditions may be different for both 2D and 3D models. Therefore the response tends to vary. Although 2D model response is more conservative as compared to 3D modelling, these differences could be noteworthy and may significantly affect the overall probability of failure of a structure under fire conditions. A few researchers have compared structural response obtained from different structural modelling (Quiel and Garlock, 2010, Flint *et al.* 2006). Quiel and Garlock (2010) investigated the performance of 2D and 3D finite element models of column, girder and filler beams for one fire scenario. They observed that a 3D model of girder produced lower deflections compared to a 2D model. This outcome is valid due to the additional stiffness that was provided by the continuous slab and adjacent elements of the structure. The results clearly state the difference in response but do not include the impact the differences have on the probability of failure (or structural response). It is also observed that no work has been done so far in quantifying the risk derived from the analysis of different structural configurations. The quantification of this difference is important because it provides information about the risk involved in the design solution due to the choice of the structural model. A noticeable variation in the probability of exceedance of structural response from different structural configuration may significantly affect the probability of failure which invariably governs the cost to reduce the failure risk as shown in Figure 6.1. Therefore, this chapter compares the probability of exceedance of structural response given multiple fire scenarios derived using different structural models, 3D and 2D.

6.2 Structure modelled

The structure used in this chapter is different from the structure used in the case studies in Chapters 4 and 5. The earlier structure, which was a typical NZ office building constructed in 1988, is difficult to model to compare the response of the 2D model of the beam with a 3D structure model. In actual secondary beams are connected to a primary beam intermittently. However, during the modelling of a primary beam as 2D element, the actual load which primary beam was going to take is converted into uniform distribution load, which is not the case in the 3D structure modelling. The beam used effectively formed part of the line of

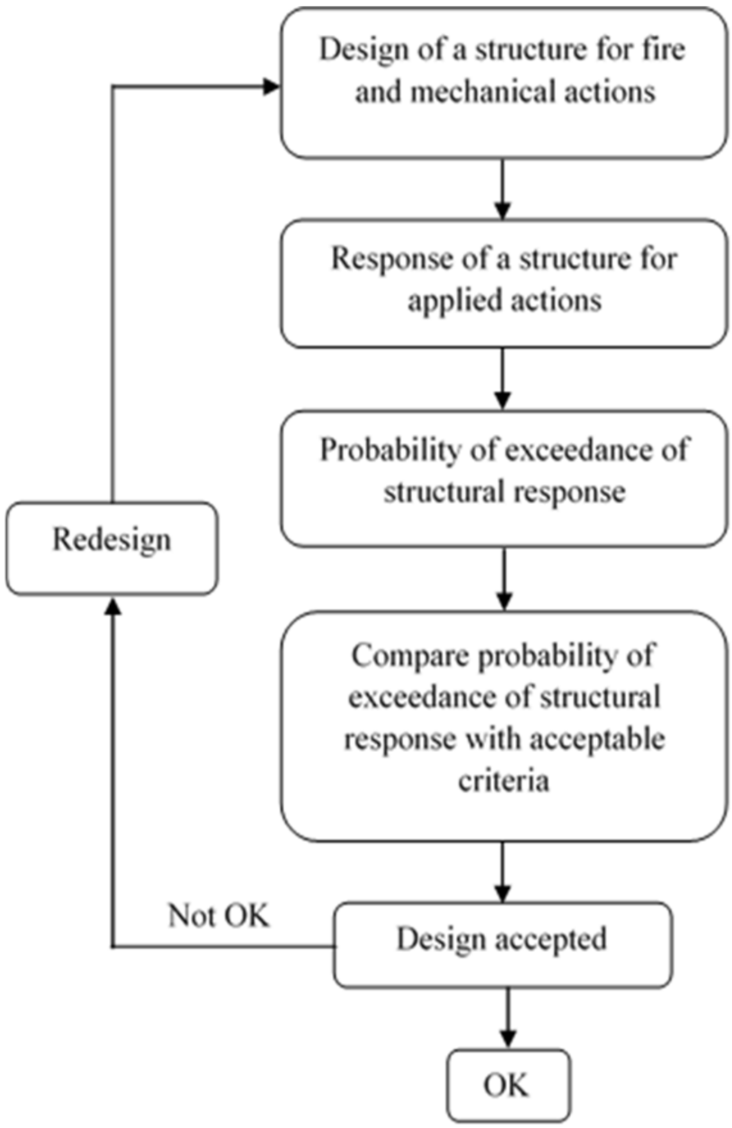


Figure 6.1: Probability based design

symmetry of the building. Thus a reasonable simulation of 3D behaviour may have required the entire floor to be modelled, although half or quarter of the floor can be modelled but then it increases the computation time also the boundary conditions (pin-pin and fixed-fixed) are to be verified for comparison with the 3D model. Also, the earlier structure does not produce membrane action due to the different arrangement of the primary and secondary beams because the slab is spanning between the secondary beams and secondary beams are transferring the loads to primary beams through a point load. It is an old office building when the concept of tensile membrane action was not popular which can substantially increase the ultimate life of the structure by accommodating excessive deflections due to its tensile membrane action without letting the structure failed. Therefore, a more modern building structure is considered here which is based on the latest design strategies and their 2D structure model should replicate the approximate response under similar loading conditions when compared to 3D structure model. Therefore, the structure used in this study was an 8-storey office building with a central covered atrium taken from a Steel Construction Institute (SCI) document called "Comparative structure cost of modern commercial buildings" (Hicks, 2004). The building is 60 m by 45 m with an atrium of 30 m by 15 m. Although the atrium has a mechanical smoke extraction system, it was not considered in this study. The building had an exterior glazed facade with its floor plan as shown in Figure 6.2. The structural configuration of the building is composite construction involving concrete flooring with steel beams and steel columns. The concrete slab was 130 mm thick, reinforced with A193 mesh. The beam end connections and mesh were detailed to achieve the necessary robustness to BS 5950-1:2000 (BS 5950-1:2000).

The applied load included the weight of the slab, decking, raised floor, services and partitions, totalling 4.61 kN/m^2 and imposed load of 3.5 kN/m^2. Load combinations were chosen from the

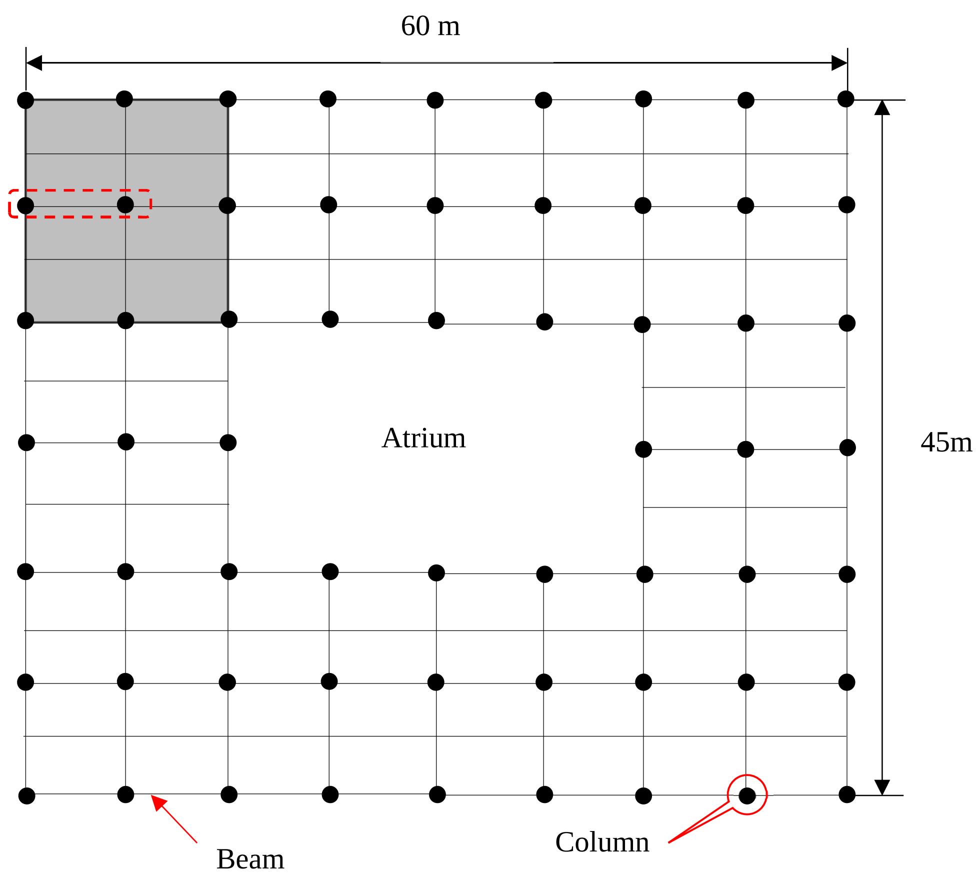


Figure 6.2: Floor plan of office building (Hicks, 2004)

Eurocode both for the ambient condition and in fire. The building was not sprinkler protected; all beams were left unprotected whereas columns had 90 min applied fire protection, using a gypsum plasterboard of density 800 kg/m^3, specific heat capacity of 1700 J/kgK and thermal conductivity of 0.2 W/mK. The building also had sealed aluminium polyester powder coated double glazed glass facade. There was no openable ventilation in the compartment, however, it was assumed that some breakage of glass will occur during the fire. The floor and roof were made up of light-weight concrete and all walls were of plasterboard.

6.3 2D Modelling

For the purposes of the 2D analysis, the highlighted beam in the shaded part of Figure 6.2 was modelled isolated with effective concrete slab as shown in Figure 6.3. The length of the beam was 7.5 m. The effective width of the slab was calculated from the minimum of ¼ of the beam length (0.25 x 7.5 m = 1.875 m) or beam to beam distance (3.75 m). The beam (305 x 165 UB40) acting compositely with a concrete slab of 130 mm thickness was modelled using VULCAN software. The fire load on the effective width of slab was the tributary load acting on the beam i.e. 12.02 kN/m^2. The beam was unprotected and was assumed pin supported at both ends based on the study described in Section 4.3. The beam is modelled in a 3D space but the support conditions at the ends of the beam and slab produces a 2D response of the beam. Composite action and the continuity of the slab were all considered in the modelling.

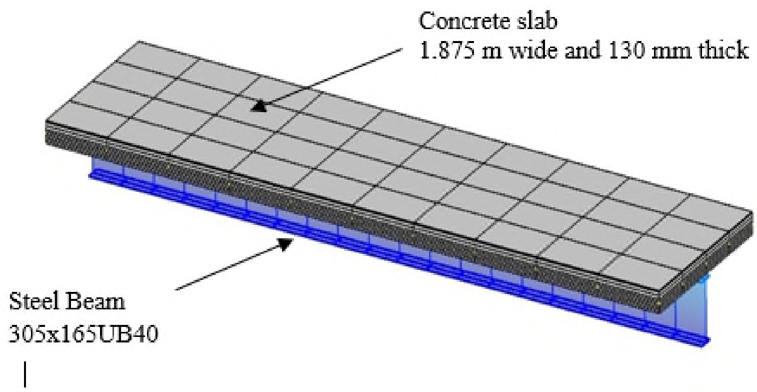


Figure 6.3: 2D Vulcan model

6.4 3D Modelling

The floor plan of the structure, as observed from Figure 6.2 is large. Therefore, to reduce runtimes, only the top left quarter of the structure was modelled in the 3D analysis as shown in Figure 6.4. The columns were fixed at the bottom and only restrained laterally at the top to avoid twisting moments in the column. However, the presence of columns in the 3D structure modelling does bring flexibility in the response and produces more realistic behaviour. Area load was applied on the slab which transfers the load to the beams and eventually to columns. Following the 2D analysis, the beams and slab were unprotected while columns were protected for 90 min fire resistance. Only the highlighted area in Figure 6.4 was exposed to fire at the bottom storey, with the rest of the structure remaining at ambient temperature. The fire compartment was considered to be located at the first floor of the building. As such loads from the storeys above were also applied on the columns.

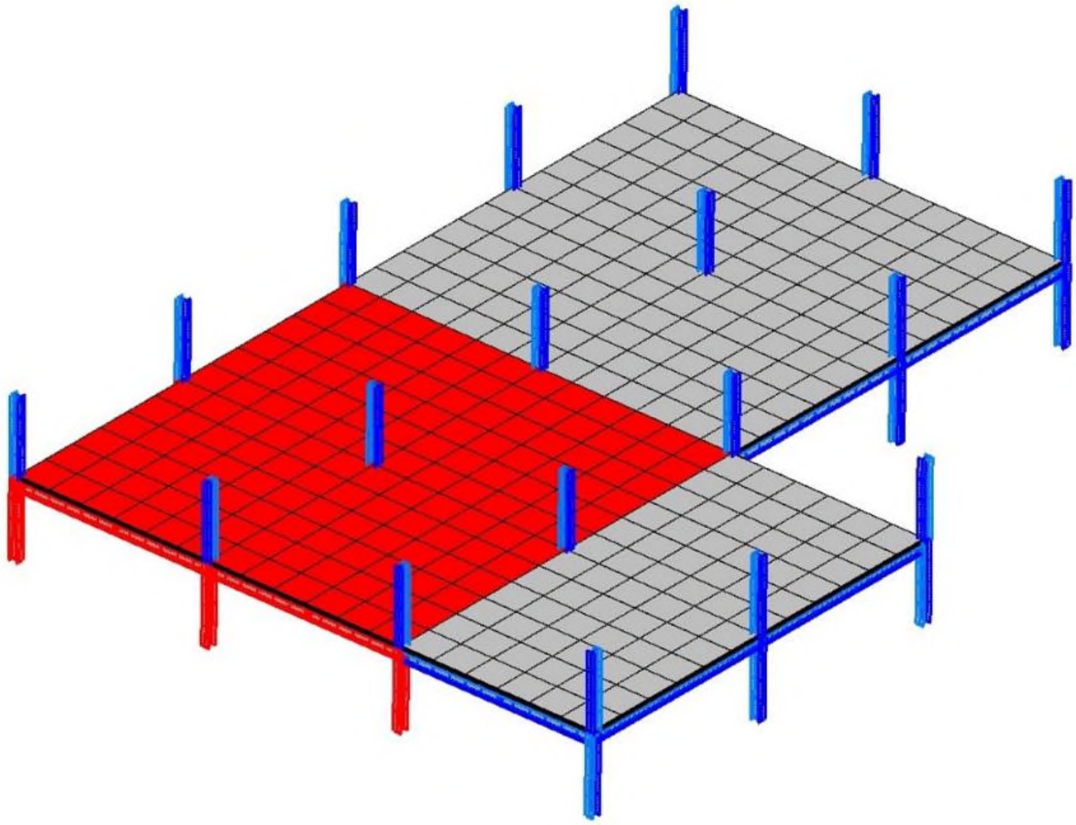


Figure 6.4: 3D VULCAN model

6.5 Fire modelling

The highlighted area in Figure 6.4 shows the compartment boundaries. It was 15 m x 15 m in size. The two outer walls were fully glazed along with 0.6 m plasterboard sill. As ventilation was treated as variable, the overall thermal inertia of the compartment 'b', which depends on ventilation area, also varies (following Eurocode 1 Part 1.2 Annex A) as shown in Equation 6.1. 'b' was calculated using the material properties of each wall (b_j, thermal inertia of wall j and A_j area of the wall j), floor and ceiling and the total area of the compartment (A_t) along with the ventilation area (CEN, 2002). Due to the variation in the ventilation (A_v), as per the Equation 6.1, 'b' is calculated as a different value for each size of ventilation.

$$b = (\sum(b_j . A_j))/(A_t - A_v) \tag{6.1}$$

Since this study was performed for an office building, the 80% fractile fuel load (q_{fk}) for office occupancy as given by the Eurocode is 511 MJ/m^2 with COV 0.3 following a Gumbel distribution. The ventilation of the compartment (A_v) varied as per an equation given by the JCSS code (Vrouwenvelder, 1997) $A_v = A_{vmax} (1- \zeta)$. The parameter '$\zeta$' has a lognormal variation with mean 0.2 and standard deviation 0.2, where A_{vmax} is maximum ventilation area. On the basis of structural information and variation of fuel load and ventilation factor, a catalogue of fire profiles was generated using Monte-Carlo simulations, as shown in Figure 6.5.

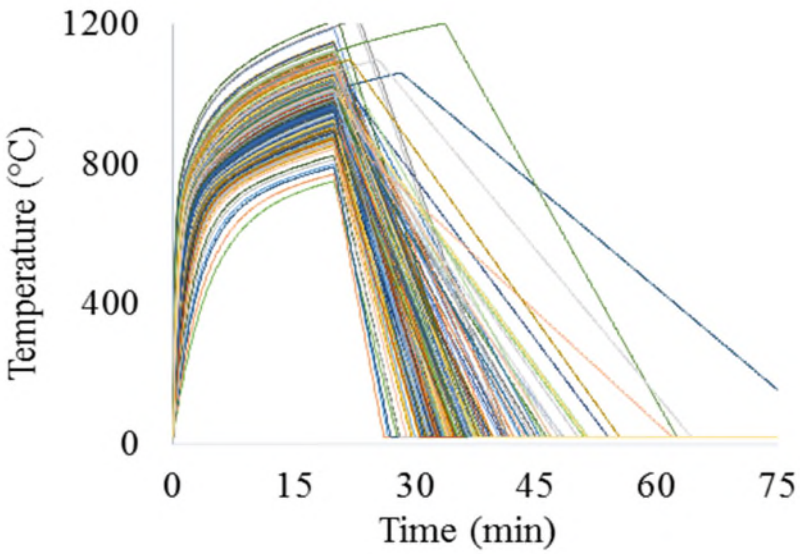


Figure 6.5: Temperature-Time profiles

Both 2D and 3D structural models were exposed to the generated fire profiles and their responses were recorded. For the analyses in this chapter, maximum deflection was selected as the structural response parameter (EDP). Mid span deflection of the beam may not be a direct indication of failure, but it was considered as a useful measure of similarity between global responses of models. It is a consequence of the reduction in strength and stiffness of a member. A limit was defined as span/20, which is considered appropriate as it is a failure criterion of structural elements in the by BS 476 standard (BSI, 2009).

6.6 Results and Discussion

Figure 6.6 shows the median plots of the results of beam deflections in the 3D and 2D models. The results are shown with variability to show the range of deflections in each model, as a result of the variation in the fire inputs. The beam deflects along with the concrete slab when exposed to fire but regains some of the deflection during cooling as illustrated in Figure 6.6. The maximum deflection of the beam is recorded in both model arrangements. The 3D models experience slightly less deflection in comparison to the 2D cases. This is primarily due to the stiffness of the model. The 3D model possesses more stiffness, hence less deflection due to the adjacent cooler regions of the structure and the continuous concrete slab. Another factor which has caused some of the reduction in the deflection of 3D model is tensile membrane action in the slab (Wang, 1996; Bailey, 2004). The double curvature of the two-way bending slab induces tensile membrane action while that effect is not so easily captured in the 2D model. In the 3D model the columns are pulled inward due to increased deflection of the floor and beams as shown in Figure 6.7. The 3D model also has a continuous concrete slab with steel beams which aid in redistributing loads from a severely heated and weakened column to nearby columns, whereas in the 2D model the supports are pinned, and therefore no movement in support is allowed and complete force or moment is taken by the beam, resulting more deflection. Therefore, the effect of the 3D composite floor system produces more realistic results than the 2D model. The results are comparable with the results from Quiel and Garlock (2010) and have similar difference in the deflection from 2D and 3D structural modelling.

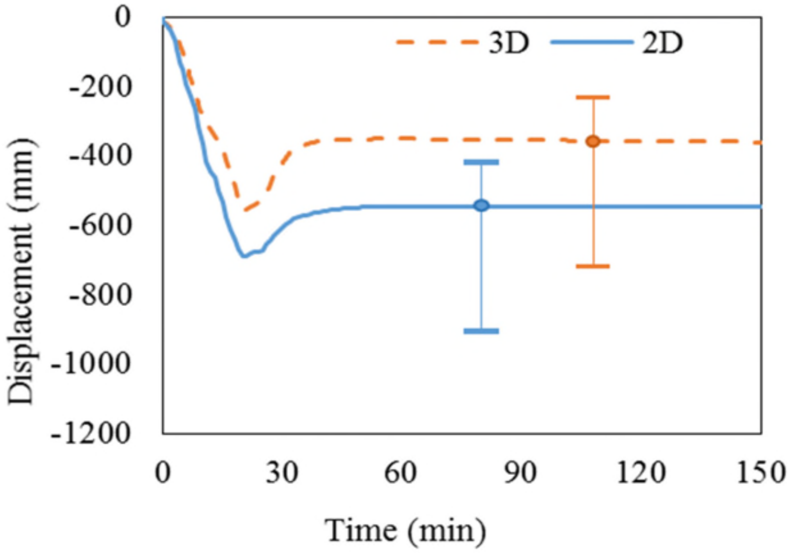


Figure 6.6: Comparison of displacement from 2D and 3D models for one analysis

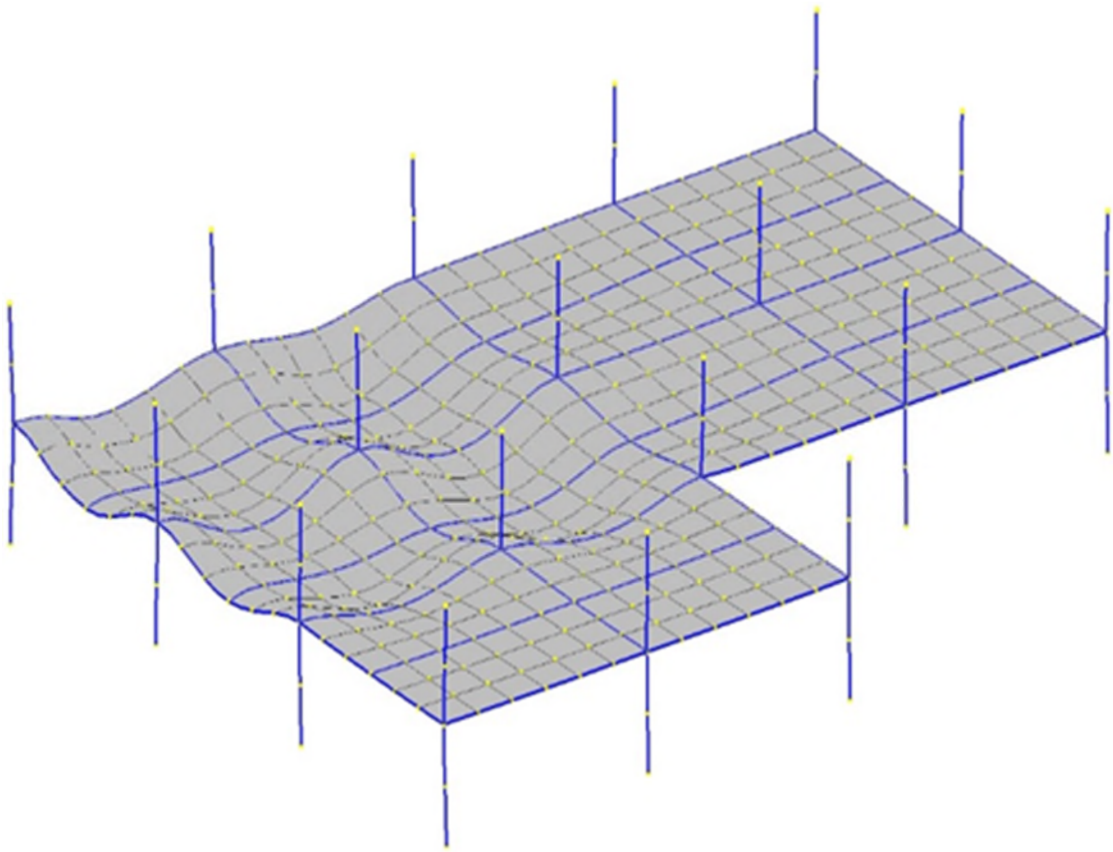


Figure 6.7: Deflected shape of 3D-model

In this study, the structure was pushed to extreme limits and did experience runaway failure. Figure 6.8 shows the runaway failure of one 2D analysis case, as a sharp downward deflection is observed in the deflection curve. This is mainly due to the sudden decrease in the stiffness of the beam. For a similar fire scenario, the 3D model remains stable and does not experience runaway failure. The variation in the 2D model response and the 3D model response for a fire is also shown in Figure 6.6. The bars in the graph indicate the maximum and minimum response from both the models. It is observed that though there is a difference in responses, the variability in responses has a similar range for both models.

The average time taken for the completion of one 2D analysis (which is of 150 minutes fire exposure) was four hours, whereas the 3D analysis took an average of 3-4 days to complete because of the increased numbers of nodes, beams and slab elements, as well as satisfying convergence criteria for nonlinear analysis. As a result of the length of the 3D analysis runtimes, the results only compared 50 analyses each for the 2D and 3D models. This can be improved by performing more analyses to cover a broader range of fires.

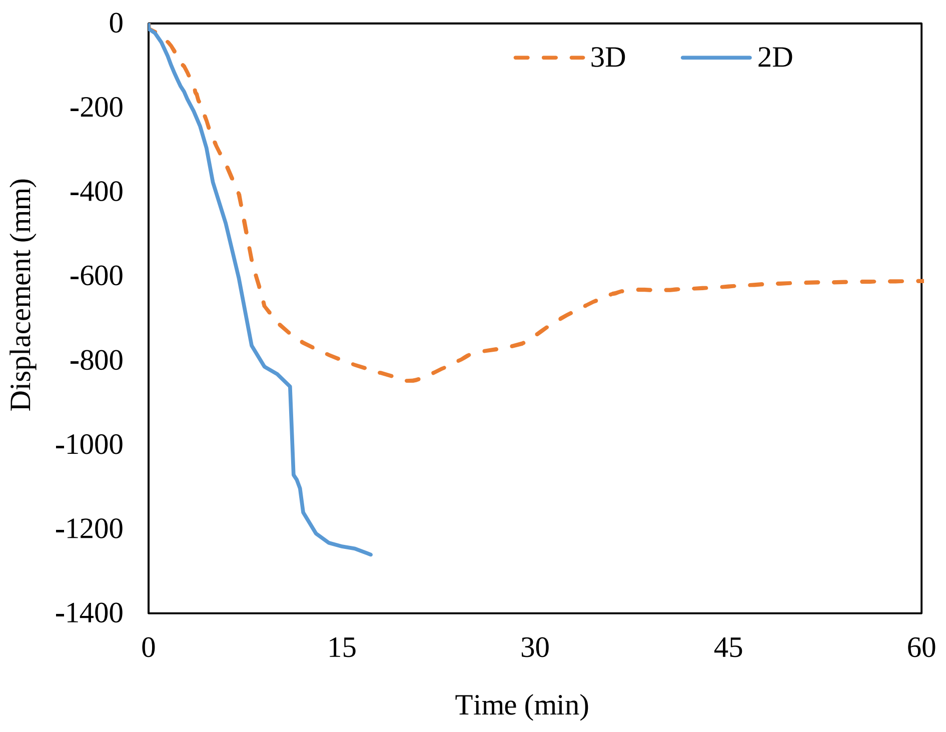


Figure 6.8: Comparison of runaway failure displacement from 2D and 3D models.

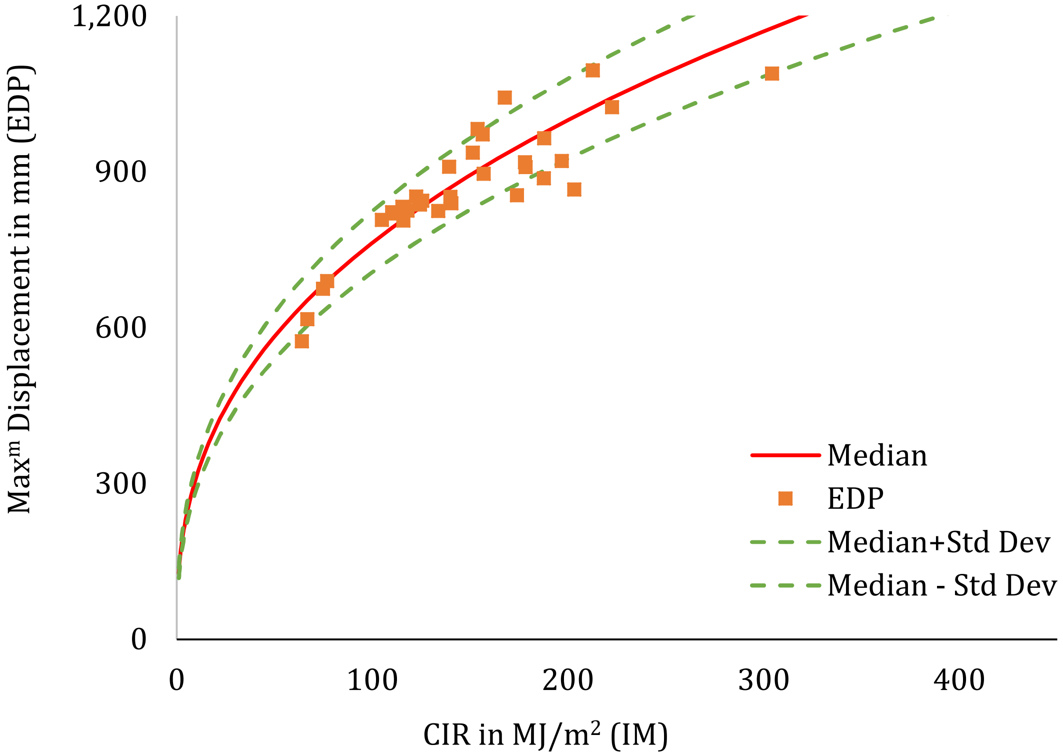


Figure 6.9: Raw EDP values of 2D model with median and standard deviation.

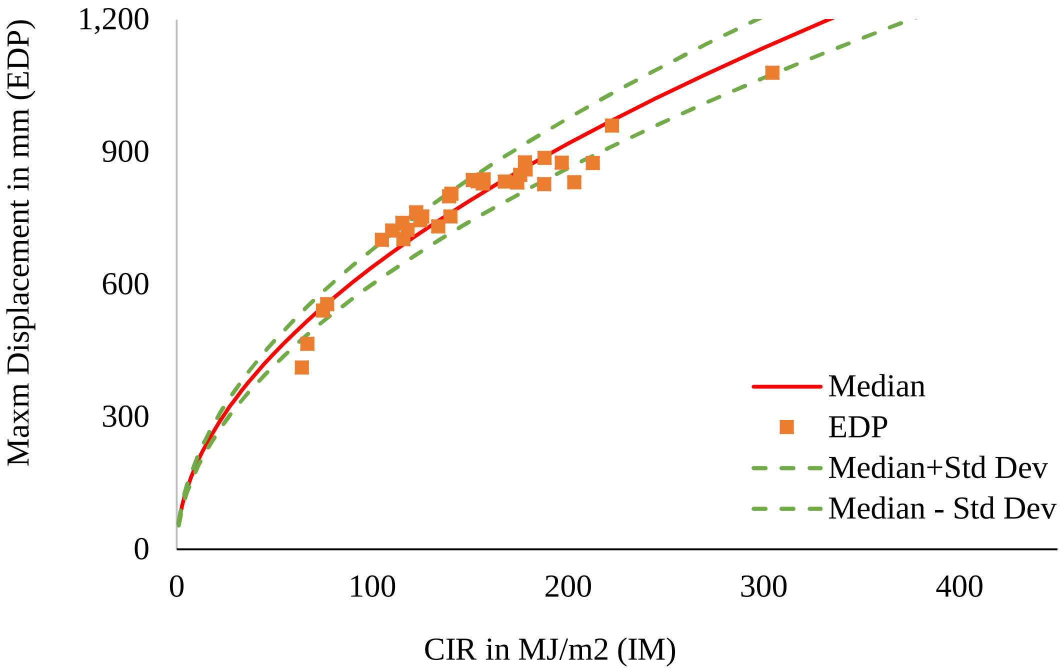


Figure 6.10: Raw EDP values of 3D model with median and standard deviation.

CIR-maximum displacement data from each analysis were collected and plotted, Refer Figure 6.9 and 6.10. Here a linear relationship is assumed between ln(CIR) and ln(max^m disp) to evaluate the conditional probability P(EDP|IM) for both 2D and 3D analysis. Furthermore, λ(EDP) is calculated based on the evaluated P(EDP|IM) and λ(CIR). The probability of exceedance of the fire severity measure (i.e. CIR) is shown in Figure 6.11. The approach is similar to cloud analysis of earthquake engineering, which has been validated in structural fire engineering by Shrivastava *et al.* (2017).

The probability of exceedance of structural response (i.e. maximum displacement) is compared from both structural modelling approaches as shown in Figure 6.12. It is clearly visible that the 3D model produces less displacement in comparison to 2D model, hence reduces the probability of exceedance of maximum of displacement.

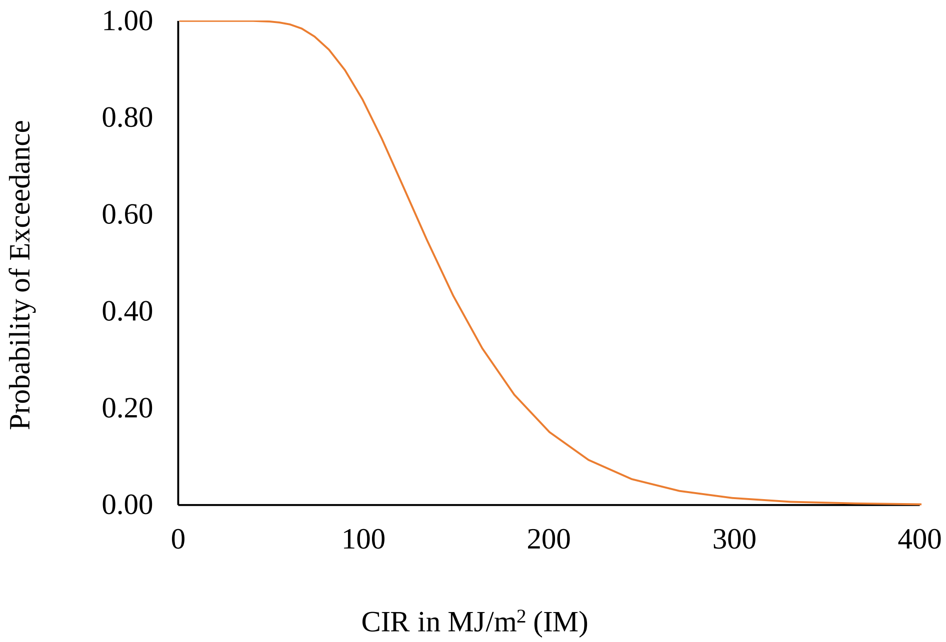


Figure 6.11: Probability of exceedance of CIR

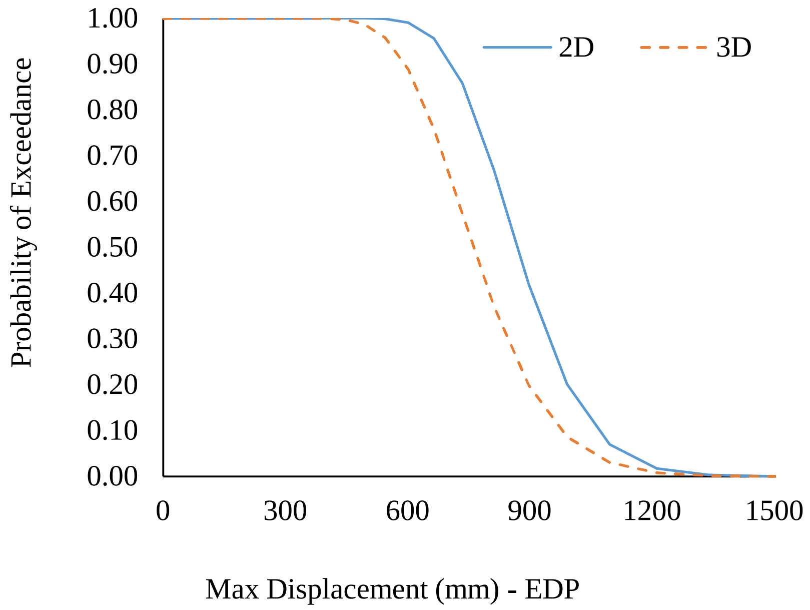


Figure 6.12: Probability of exceedance of maximum displacement

6.7 Conclusion

This chapter evaluated different ways of finite element modelling and compared the efficiency of computation with results. The deemed motive of this comparison was to investigate the effect of structural configuration on the probability of failure or structural response. This was demonstrated through a virtual case study performed on an office building. The response of a beam is observed in two different structural configurations; first as an isolated beam modelled in 2D and then as part of a larger structure in a 3D model, all exposed to fire.

It is observed that 3D modelling produces smaller deflections and comparatively less probability of exceedance of a level of displacement. It is also important to consider the computation time when selecting the structural configuration for modelling. 3D analysis may produce more accurate results but could be time-consuming. It can be concluded that if a member is designed as isolated then it will produce conservative results.

7 UNCERTAINTY IN STRUCTURAL RESPONSE

Uncertainty has a major influence on the assessment of the reliability of a structure. Uncertainties are classified conventionally into two categories: aleatory and epistemic uncertainty. Aleatory uncertainty arises from the inherent randomness in nature. Fuel load, occupancy load and geometry of the room are examples of parameters which possess inherent randomness. The randomness of these parameters may be represented by probability distributions. Epistemic uncertainty on the other hand is due to the lack of knowledge. This uncertainty can be reduced by increasing the knowledge about the tools we use for structural and thermal analyses (e.g. fire models and structural analysis models).

The randomness associated with post-flashover fire development factors such as fuel load, ventilation, room geometry and surface lining are classified under aleatory uncertainty. Many researchers have looked into the variation of these parameters individually, but no one has investigated the collective randomness in the above mentioned four key elements together to then quantify the associated aleatory uncertainty. Nowadays the design of an office building has become more architecturally challenging and the increased use of glazed facades implies a potential increase in opening areas leading to a higher probability for more fuel controlled fires (due to the presence of excess oxygen for burning). This has raised concerns about the suitability of the readily available distributions to new buildings. Therefore, there is a need to update the probabilistic distribution of these key elements through new surveys or combining available surveys.

The advantage of new surveys of surface lining properties, compartment area and ventilation area is not only limited to updating the distribution type and mean value for use, but it is also helpful in obtaining more accurate annual probability of severe fire and structural response of the building. This information is very useful for the assessment of a building for insurance and resale, which makes the insurance company/buyer aware of the future potential maintenance cost of the structure due to probable fire event which may occur during its lifetime. This may also be useful in determining the vulnerability of the building stock of a community, city or country to large fires. This will also serve as a tool to quantify the repair work required to upgrade building to reduce fire risk.

Once the distributions of post-flashover fire development factors have been re-established they may then be used to generate new fire profiles. Fire models will then recreate fire scenario based on the input data and generate associated temperature-time curves. Post-flashover fire models were discussed in Section 2.3. Temperature-time profiles generated from various fire models using the same input are compared in literature (Barnett, 2002; Pope and Bailey, 2006). This indicates that there is uncertainty in predicting temperature –time curve if we use different fire models. However, how much uncertainty propagates in predicting structural response due to fire model, i.e. epistemic uncertainty, has not yet been addressed in any research. This uncertainty due to fire model can significantly affect the reliability of the structure (annual probability of structural response). Therefore a careful selection of fire model is required to represent the probable fire scenario which could possibly occur in a structure.

Based on the above research gaps, the objectives of this chapter of the thesis are:

1. Produce an updated probability distribution for each key element of post-flashover fire, such as fuel load, ventilation, room geometry and surface lining.
2. Quantify uncertainty in structural response due to the random nature of the post-flashover fire development factors.
3. Quantify uncertainty in structural response due to use of different fire models.

7.1 Aleatory uncertainty

The quantification of aleatory uncertainty is performed by considering the probabilistic distribution of the inputs and the variation in structural response, as calculated by the dispersion of response, which indicates the degree of impact of the randomness of input parameters on the structural response, see Figure 7.1. This study investigates fuel load, ventilation, room geometry and surface lining as random parameters and discusses their distribution in detail.

As discussed in Section 2.2, Compartment geometry, ventilation and surface lining are significant contributors to the fire development in addition to fuel load. At the planning stage room sizes are decided and compartment dimensions are confirmed by the fire engineer to ensure designs satisfy code requirements. Although compartment information is useful for fire growth, it varies tremendously with geographic location, for the same occupancy, cultural area and building plan. Therefore it is nearly impossible to specify one value of the compartment floor area for office occupancy for a particular region, more so globally. Nowadays the demands of architecture of office buildings is rapidly changing adopting more glazing, which indicates that the maximum ventilation available in the compartments is increasing. Similar to ventilation, the variation in material properties of office compartment structures is large due to the use of timber, steel, concrete and other building materials. The thermal inertia of the compartment varies tremendously because of the use of different surface materials. Therefore it is important to understand the modern trend of office buildings in terms of compartment geometry, ventilation and surface material. There is a need for new surveys which provide average opening factors, thermal inertia and geometry of the compartments in modern office buildings.

A survey was performed in Christchurch that gathered information about compartment geometry, ventilation and material properties of surface linings used in office buildings to identify a mean value of each element and its variation. These office buildings were a mixture of university offices and general offices in the city area. A total of 12 buildings with 79 compartments (approx. 800 rooms) were surveyed with total floor area of 51707 m^2. A compartment was comprised of several office rooms and these compartments were separated by fire rated walls as per code requirements. The survey was based on architectural drawings of the buildings and physical examination of the buildings. Since window schedules and surface lining material information for all the buildings were not available, only 3 buildings with 28 compartments (approx. 250 offices) were surveyed for ventilation and surface lining. For each compartment, the wall dimensions, window dimensions and thermo-physical material properties of each lining was recorded.

Fuel load has a significant variation in its mean value as discussed in section 2.2. Several attempts have been taken in the past to produce a more robust value of fuel load. Surveys performed at the same geographic location within the same cultural area also produced different results, underlining uncertainty due to different survey techniques, different approaches of statistical interpretations of results, number of observations and the use of the office facility. The difference is too large to ignore. Therefore, it is helpful to produce one distribution for each region and occupancy type which embodies all the recent surveys performed so far in that region for similar occupancy. In this study, a new mean value and distribution type is proposed for office buildings in New Zealand (similar information may be provided for other regions and occupancy types) by combining several individual surveys of New Zealand. This task will eradicate the dilemma of when to use which survey result in design.

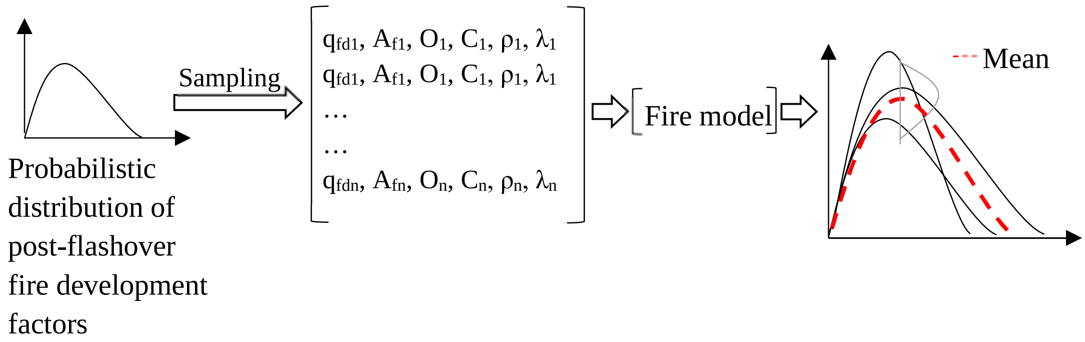


Figure 7.1: Aleatory uncertainty due to random nature of post-flashover fire development factors

7.2 Survey results of Post-flashover fire development factors.

Fuel load

A collection of surveys performed recently on office buildings and available in literature are summarised in Table 7.1. These surveys were conducted at different times, different geographic locations and different cultural areas and hence justify the differences in average fuel load density for office buildings. In Table 7.1 the boxed observations were not reported in the raw data (e.g. 1991, US-4 region by Korpela), and were computed as the average of the total number of observations. For data for which standard deviation is unavailable, 0.3 is used as a coefficient of variation based on Eurocode (CEN 2002). Therefore if we are producing one combined results for every region then average number of observation is 241. Therefore 241 can be used in place of a missing number of observations and 0.3 COV is used in place of a missing number of standard deviation as shown in Table A.1 in Appendix A.

Table 7.1 summarizes surveys performed in a few regions around the world. In New Zealand, four surveys were performed from 1984 to 2008 which provided average fuel load value ranging from 436 to 950 MJ/m^2. New Zealand data were combined together to produce one result. Therefore as discussed above the missing observations in the New Zealand data were filled by the average value of the number of observations (i.e. 4). The error is calculated by substituting maximum and minimum value at the place of missing number and variation is observed. Similar to the combined mean of every region it was observed that there was a 1% error in the combined mean of fuel load for New Zealand due to this assumption, See Table A.2 in Appendix . This error was small therefore ignored here in calculation.

The New Zealand data were combined together to produce one distribution of fuel load (q_{fd}) for New Zealand. The combination technique used here was basic, and the results can be refined by using advanced combination techniques. Here, the mean ($\bar{\hat{x}}$) distribution and standard deviation ($\bar{\hat{\sigma}}$) of distribution are calculated from Equations 7.1 and 7.2 respectively. Here n_i is the number of observation of i^{th} survey, x_i is the mean of i^{th} number of observation.

$$\bar{\hat{x}} = \frac{\sum_{i=1}^{m} \bar{x}_i . n_i}{\sum_{i=1}^{m} n_i} \tag{7.1}$$

$$\bar{\hat{\sigma}} = \frac{\sum_{i=1}^{m} (n_i - 1).\sigma_i{}^2 + n_i\{\bar{x}_i - \bar{\hat{x}}\}^2}{\sum_{i=1}^{m} \sum n_i - 1} \tag{7.2}$$

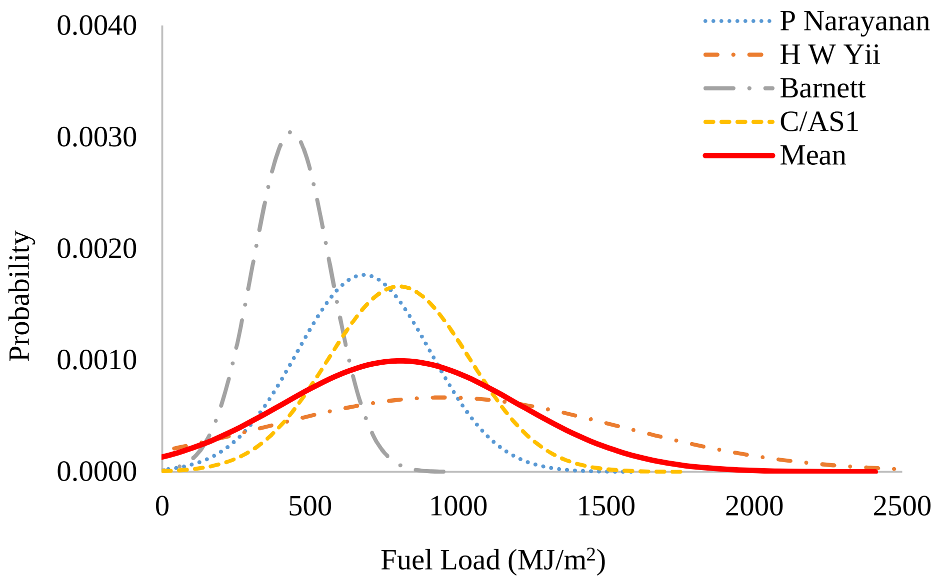


Figure 7.2: Probability distribution of fuel load

Table 7.1: Collection of Fuel load survey over past 50 years.

Year	Region	No of Observation	Mean	Standard Deviation	Author
1968	US-1	453	348	-	Bryson/Gross
1957	US-2	82	1270	-	Ingberg
1995	US-3	6	1298	-	Caro
1991	US-4	-	1120	-	Korpela
1986	US-5	419	555	285	Culver CIB W14 report
1986	US-6	625	525	355	Culver CIB W14 report
1995	NZ-1	5	681	226	P Narayanan
2000	NZ-2	7	950	598	H W Yii
1984	NZ-3	1	436	-	Barnett
2008	NZ-4	-	800	-	C/AS1, NZ building
2008	France	61	657	290	Thouvoye
1999	Finland	165	730	219	Korpela
1986	Europe-1	414	420	370	CIB W14 report
1986	Europe-2	-	410	330	CIB W14 report
1986	Europe-3	-	330	400	CIB W14 report
1986	Swedish	-	411	334	CIB W14 report
1993	Swiss-1	-	500	-	CIB 1993 report
1993	Swiss-2	-	600	-	CIB 1993 report
1970	UK-1	93	400	-	Baldwin
1991	UK-2	-	698	-	Butcher
1993	UK-3	801	418	301	Melinek
1993	India	388	348	262	Sunil/Rao
2011	Canada	103	557	286	Ehab/James
1999	AU	-	800	480	FCRC

Figure 7.2 illustrates the combined normal distribution of fuel load for New Zealand. Similarly the process can be replicated for other regions to produce one distribution for each.

As observed from Figure 7.2 the variation is too large to ignore. Therefore, providing one distribution and mean value of fuel load for each major region as well as one global distribution (applicable to any region) is helpful for precise probabilistic prediction of fire temperature in any compartment, but not realistic, as local variations may either be underestimated or overestimated (in comparison to the globalized distribution).

Table 7.2: Floor area survey results performed on office buildings in Christchurch, New Zealand

Year	**Region**	**No of Observation**	**Mean**	**Standard Deviation**	**Author**
1995	NZ-1	5	681	226	P Narayanan
2000	NZ-2	7	950	598	H W Yii
1984	NZ-3	1	436	131	Barnett
2008	NZ-4	4	800	240	C/AS1, NZ building
Total	NZ	-	805	424	Combined

Compartment geometry

Table 7.3 presents the results of the survey performed on 79 compartments as mean values of the compartment floor area with their standard deviations. The statistical investigation was carried out to analyse the surveyed data to obtain probability distributions, mean compartment floor area and standard deviation. Firstly, a frequency distribution was obtained from the data which was further utilized to fit suitable probability density functions. The Kolmogorov-Smirnov (K-S) goodness of fit test was performed to obtain the probability density function of the observed data. Figure 7.3 illustrates that a log-normal distribution fits best to the compartment floor area data.

Table 7.3: Floor area survey results performed on office buildings in Christchurch, New Zealand

Property	**No of compartment**	**Mean compartment floor area A_f (m^2)**	**standard deviation**
University office buildings	51	433.0	172.15
City office buildings	28	1085.69	669.09
Total	79	659.87	504.08

The results of the survey indicated that university offices have less compartment area when compared to city offices. Also it was observed that the variation (i.e. standard deviation) was more in the city offices than university offices. The major reason for this difference was that the university owns its properties, compartments are smaller but distributed over the height of their buildings, while different city offices may share one large compartment as the whole building is owned by one person or organisation. It has been observed that in city area due to space constraints all sizes of offices are available which varies from very small to large. In this present study the office floor area in city varies from 113 to 2663 m^2 and for university offices it ranges from 184 to 882 m^2.

Ventilation conditions

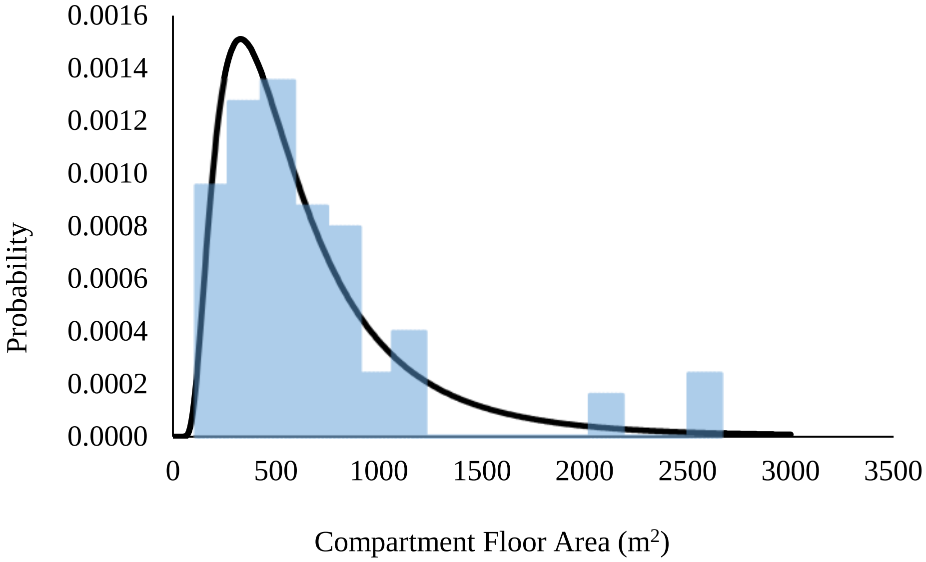


Figure 7.3: Probability distribution of floor area

Another important element governing compartment fire growth is ventilation. The ventilation of a compartment is expressed as opening factor in Equation 7.3:

$$O = A_v . \sqrt{h_{eq}} / A_t \tag{7.3}$$

where A_v= ventilation area, h_{eq} = weighted average of window height and A_t = total internal surface area of the compartment (CEN, 2002). Contrary to fuel load, there is limited data available on ventilation area, which varies widely even amongst the same occupancy. In 1976,

Culver suggested an average opening factor of 0.081 with standard deviation 0.07 for office buildings based on a survey performed in US. P Narayanan in 1995 performed a survey in New Zealand on office buildings and recommended 0.08 as the average opening factor with 0.035 as standard deviation. It was observed that both the aforementioned surveys indicate similar average opening factor even though the investigation were performed more than two decades apart (refer Table 7.4). The most recent of these studies was carried out more than 20 years ago. Buildings layouts and ventilation provisions have changed a bit since then and new surveys are needed.

Table 7.4: Collection of opening factor survey performed on office building

Author	**Year**	**average opening factor**	**standard deviation**
Culver	1976	0.081	0.070
P Narayanan	1995	0.080	0.035

The survey results performed on 28 compartments are presented in Table 7.5. A K-S test was performed on the surveyed data to identify the distribution of best fit. Lognormal distribution was found to best fit the data. The mean of the lognormal distribution was 0.103 and standard deviation was 0.061 (see Figure 7.4).

Table 7.5: Ventilation survey data performed on office building in New Zealand

S No	No of compartments	average opening factor (O)	standard deviation
Building 1	12	0.130	0.080
Building 2	9	0.100	0.020
Building 3	7	0.060	0.010
Total	28	0.103	0.061

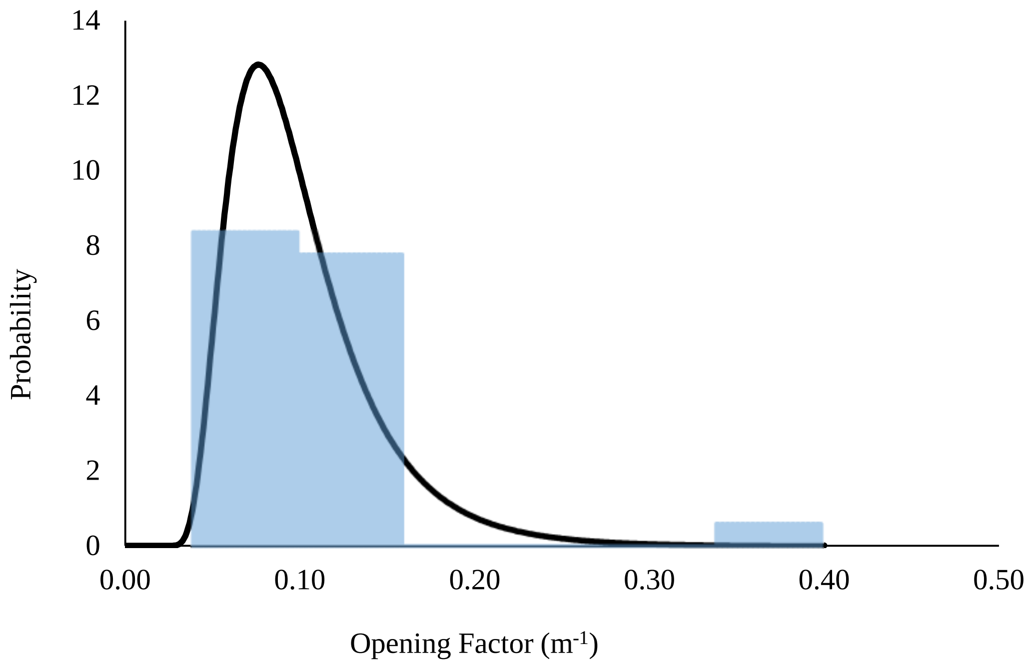


Figure 7.4: Probability distribution of opening factors for 3 building surveyed

Surface lining

The surface lining of the compartment is measured by Equation 7.4.

$$b = \sqrt{k.\rho.c_p} \tag{7.4}$$

where k = thermal conductivity, ρ = density and c_p = specific heat of the surface material (CEN, 2002). Similar to compartment geometry and ventilation, the material properties of the walls, floors and ceiling of the compartment are typically decided before construction. Therefore it is normally considered as a deterministic parameter. The thermal properties of various construction material indicate that thermal inertia of the compartment varies largely from 500 to 2100 J/m^{2}s$^{1/2}$K. In modern New Zealand office, constructions are typically of either reinforced concrete or steel-concrete composite or timber with plasterboard walls. This means sometimes two different materials are present in one compartment (e.g. floor and roof of concrete and walls of plasterboard). Eurocode 1 has suggested an equation (Equation 7.5) to for account different material properties in the compartment and provides one combined value of thermal inertia (CEN, 2002). Here, "b" is the combined thermal inertia of the compartment. b_j is the thermal inertia of the boundary surface j and A_j is the area of the boundary surface j which excludes opening area.

$$b = \frac{\sum(b_j A_j)}{(A_t - A_v)} \tag{7.5}$$

Because of the uncertainty in the material properties which propagates into the evaluation of thermal inertia of the compartment, researchers have considered it as a varying parameter in their studies (Iqbal and Harichandran, (2010); Moss *et al.* (2014); Selamet and Akcan, (2015)). The effective thermal inertia of the compartment is not only dependent on the different lining materials but also on the area covered by each material in addition to openings. A few post-flashover fire models use direct values of effective thermal inertia of the compartment ("b") but some require detailed input of each thermos-physical material property (density, specific heat and thermal conductivity). Therefore effective density, specific heat and thermal conductivity were calculated as shown in Equation 7.6. Based on the study it is observed that thermal inertia of the compartment calculated from Equation 7.5 and thermal inertial calculated by using values of combined density, specific heat, thermal conductivity in Equation 7.4 produces similar results with less than 1% error. The effective density, specific heat and thermal conductivity values were used in the subsequent fire models where individual specification were needed.

$$\rho = \frac{\sum(\rho_j A_j)}{(A_t - A_v)} \qquad C_p = \frac{\sum(C_{pj} A_j)}{(A_t - A_v)} \qquad \lambda = \frac{\sum(\lambda_j A_j)}{(A_t - A_v)} \tag{7.6}$$

The current survey results, performed on 28 compartments, revealed that normal distribution fits specific heat capacity adequately with a mean of 1110.01 J/kgK and standard deviation 32.24 J/kgK. Lognormal distribution was found to best fit both the effective density and thermal conductivity data as per the K-S test, producing a mean of 1981.11 kg/m^3 and 1.258 W/mK and standard deviation of 90.31 kg/m^3 and 0.095 W/mK respectively. The results are recorded in Table 7.6 and Figures 7.5, 7.6 and 7.7.

Table 7.6: Surface lining survey data performed on office building in New Zealand

Parameters	**Mean value**	**standard deviation**
Density (kg/m^3)	1981.11	90.31
Specific Heat (J/kgK)	1110.01	32.24
Thermal Conductivity (W/mK)	1.258	0.095

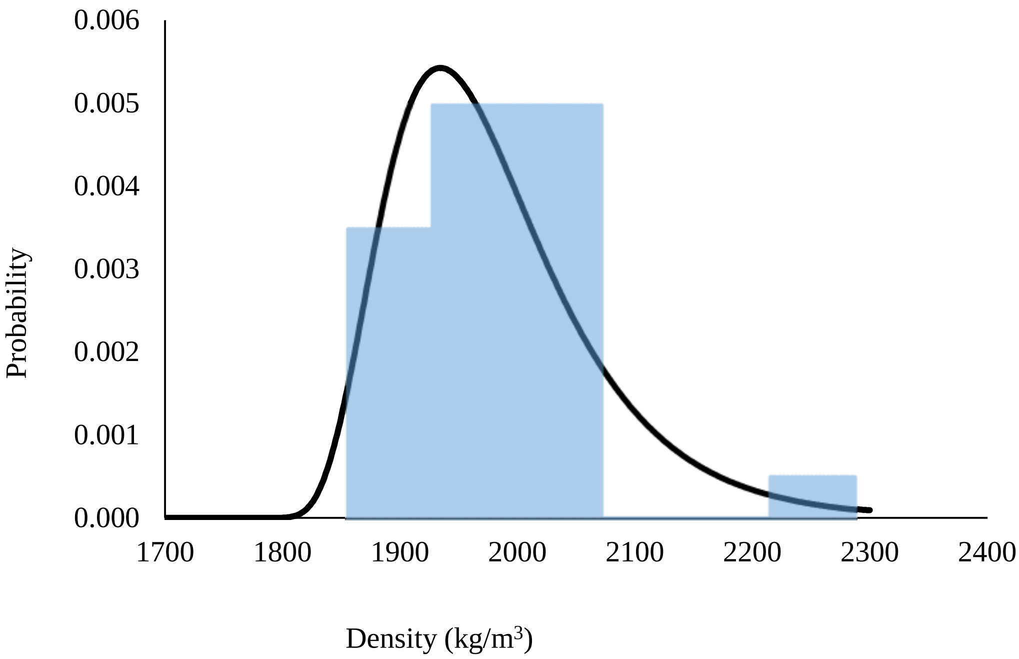


Figure 7.5: Probability distribution of Density

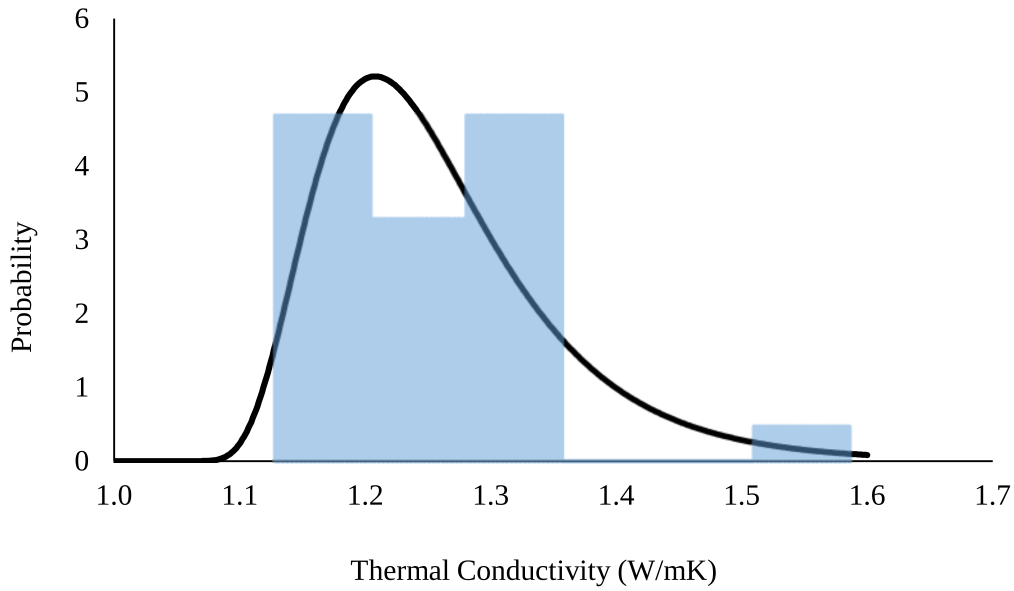


Figure 7.6: Probability distribution of Thermal conductivity

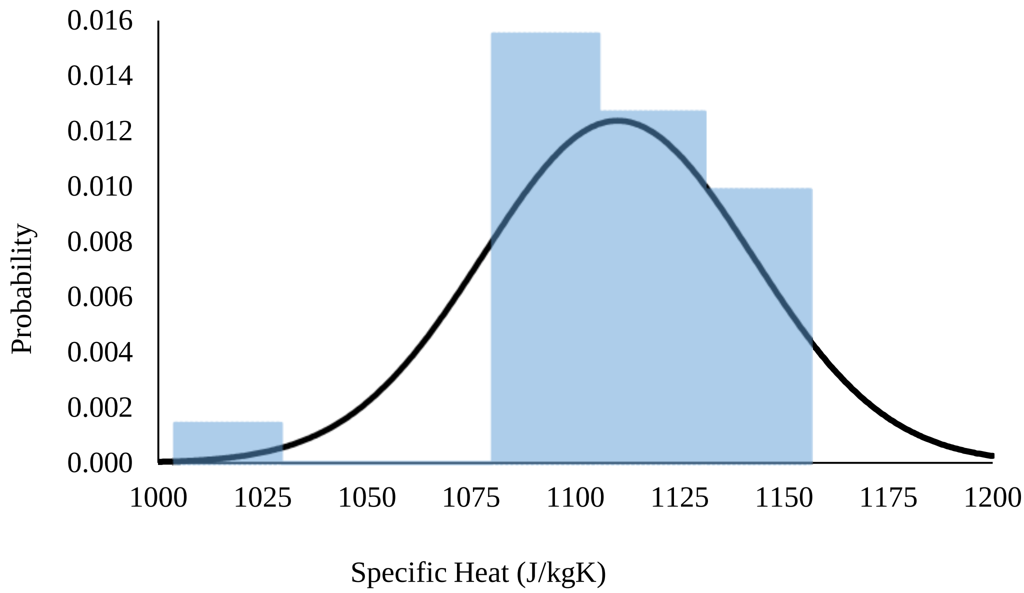


Figure 7.7: Probability distribution of specific heat

Based on the distributions of the above input parameters, a catalogue of fires can be generated. The fires were generated with the help of fire models. As discussed in Section 7.3 below single zone post-flashover fire models are preferable for structural analysis. As such the following fire models are used for the analyses. They are parametric fire, BFD curve, iBMB, B-Risk and

Ozone. These fire models produce different results for the same input because each model is based on different governing equations and assumptions. This difference in the output of the models for the same input is classified as epistemic uncertainty.

7.3 Epistemic uncertainty

Epistemic uncertainty can be reduced by having a thorough knowledge of the process and modelling. Frequent use of modelling platforms such as finite element models to analyse structures makes the propagation of epistemic uncertainty in results inevitable. In Structural fire engineering, once input parameters of the fire are established then the use of fire models is executed to replicate the fire scenario in the compartment. As shown in Figure 7.8, the use of different fire modelling platforms can produce different temperature-time relationships for the same inputs. This variation of the results from mean is called epistemic uncertainty and is quantified by evaluating the dispersion (“β”) of individual results from the mean (“μ”).

Figure 7.9 illustrates the comparison of fires from all five fire models (Parametric fire, BFD curve, iBMB, Ozone and B-RISK) with the same input values. Ozone and B-RISK have the capability of single zone post-flashover fire generation but they are software based applications that are primarily two-zone models. Their inputs need to be fed in one by one in their user interface for generation of fire and therefore not used here for the generation of 100 fires. The comparison in Figure 7.9 was for a fire generated (for the five fires) from similar input.

This study proposes to use three fire models and evaluate the uncertainty these fire models bring in during the process of generating temperature-time curves. The fire models used herein are post-flashover one zone compartment fire models. This category of fire model has been chosen in order to keep the fire modelling simple and concentrate on evaluating uncertainty.

Figure 7.8: Epistemic Uncertainty due to fire models

This work can be extended by using more fire models and of different categories to refine the results (two zones or more). Here, the research concentrate on establishing the methodology first. The fire models used are Parametric fire, BFD curve and iBMB. The process of generating the fire profile through each model and associated assumptions and governing equations are discussed in Section 2.3.

With the help of the distributions of post-flashover fire development factors models, sampling of the input was performed. Randomly sampling with LHS method was conducted and a set of input values were created. These input values were used in each fire model to generate the fires. Similarly, 100 inputs from each distribution was used to generate 100 fires for each fire models using @Risk software. The parametric fire, BFD curve and iBMB can be modelled in spreadsheet, so they were used to generate the multiple fires.. Therefore only 100 inputs were generated to produce fires from each fire model. This study can be expanded in future to refine results with probabilistic capability of the programs.

Figure 7.9 highlights the difference in the temperature profile from four fire models. FSMs such as the maximum temperature, time to reach maximum temperature, area under the fire curve and fire duration, all are different from each other. This justifies that each model runs on

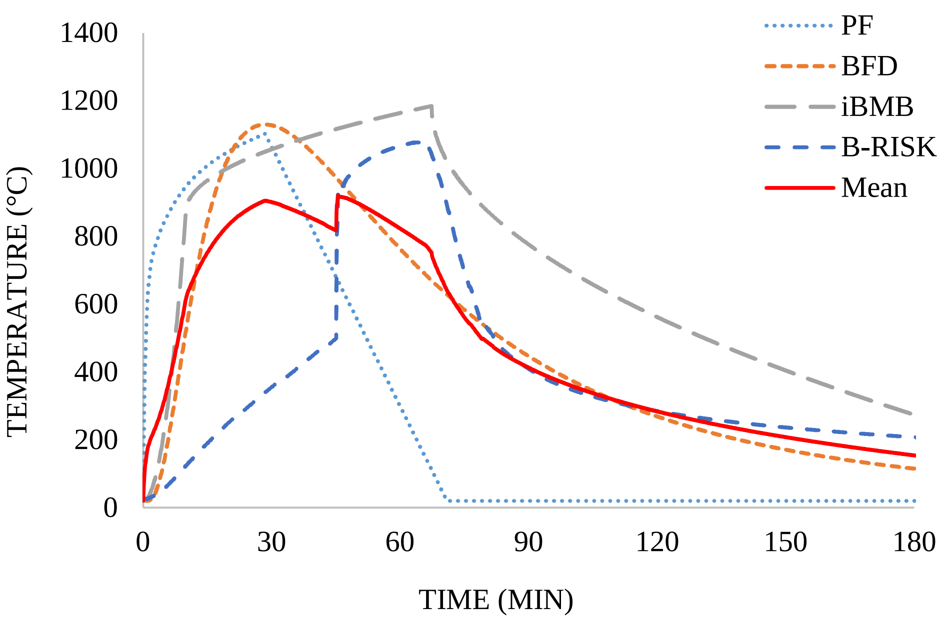


Figure 7.9: Temperature-time profiles from various fire models

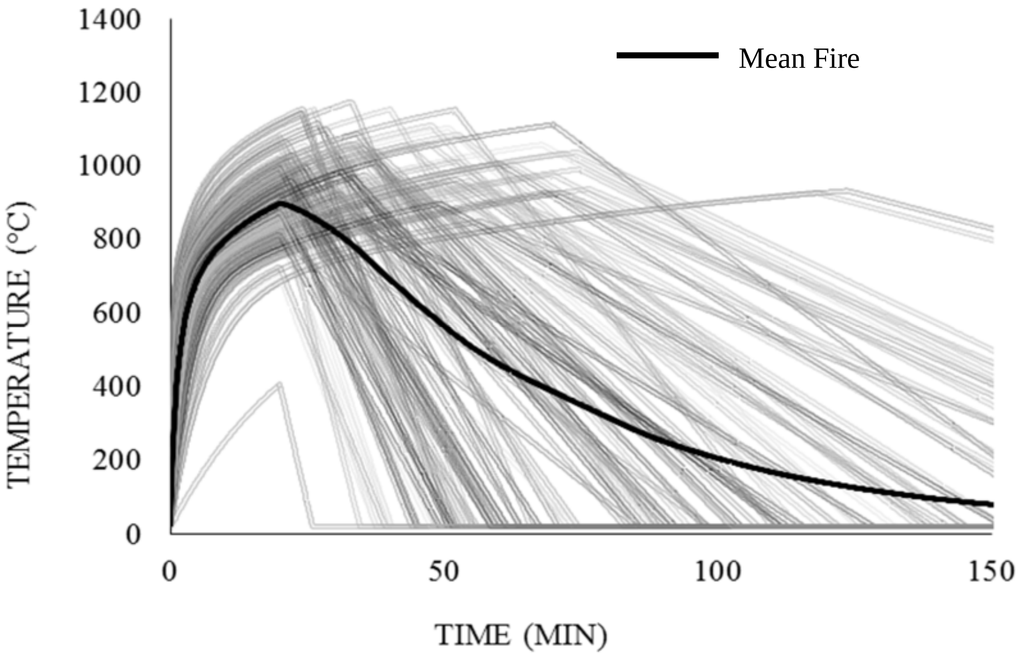


Figure 7.10: Temperature-Time profiles generated from Parametric Fire model and their mean profile

different principles and have their own assumptions. The illustration in Figure 7.10 shows a catalogue of fires generated through the parametric fire model. Similar catalogues generated from other fire models were shown in Appendix A. Figure 7.11 illustrates that since the fire models were different the resultant fires are therefore different (for the same input).

To evaluate the uncertainty in temperature-time profile due to fire models, the mean of each of the 100 fires was calculated. This means there were three mean fires, which are compared in Figure 7.11 and the mean of these three means is also shown in Figure 7.11. The variation of each mean fire from the combined mean fire provided the uncertainty of each fire model in evaluating temperature-time profile. This uncertainty was termed as β_1 and summarised in Table 7.7. It was observed that the mean parametric fire produces a lower maximum temperature and iBMB produces much higher temperatures than the BFD fire and the Parametric fire.

The difference in shape of the fire between the fire models affects the probability of generating a defined fire severity level. This information is very useful if failure probability is being calculated. Figure 7.11 illustrates that the parametric fire (PF) produces lower mean fire compared to other fire models. This indicates that the parametric fire model predicts lower compartment temperature than other fire models, whereas iBMB estimates higher temperatures. The difference in probability calculation from various fire models affects the failure probability of the structure. Therefore it is important to identify the amount of inherent uncertainty each fire model possesses in order to reasonably predict uncertainty in the structural response. Thus, uncertainty in structural response is also calculated.

After the generation of fire profiles, the process of evaluating structural response requires thermal and structural analysis. To perform structural analysis of this nature finite element packages are generally used. Every finite element program follows different principle of analysis and assumptions, which may produce different results for the same structure and input values similar to the observation from the fire models. Therefore it is another source of epistemic uncertainty. Epistemic uncertainty due to the structural modelling platform is not investigated here. The focus of this study was to evaluate uncertainty in structural response due to fire models and it can be obtained by performing structural analysis of a simple isolated element for various mean fires of each fire model. The response of a structure for each mean fire was compared and the variation of each structural response from their combined mean was termed as β_2.

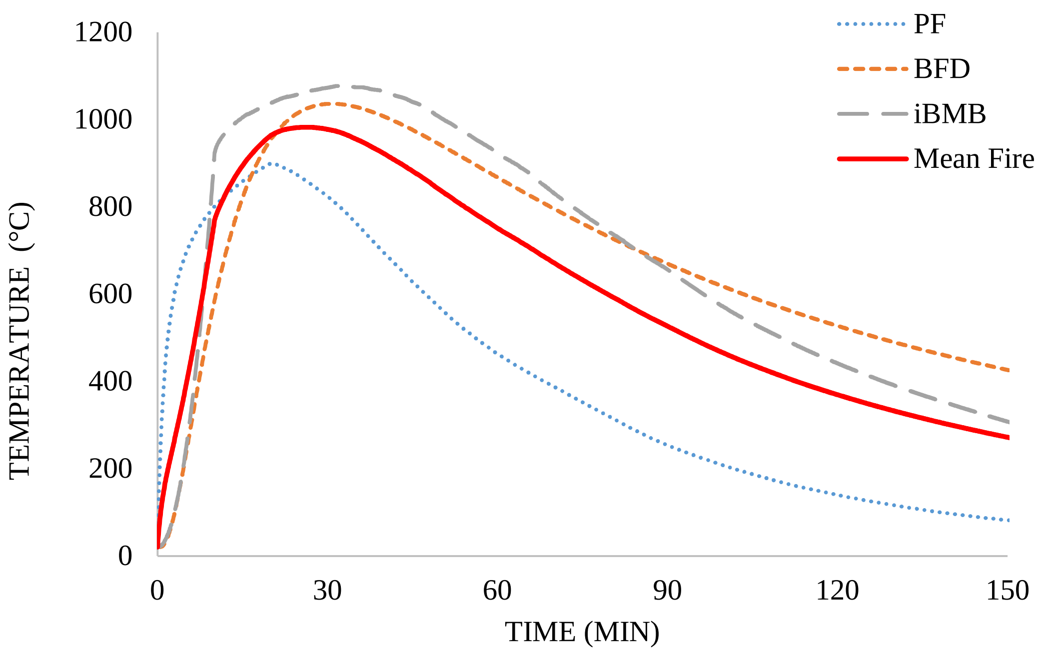


Figure 7.11: Comparison of mean fire profiles from various fire models

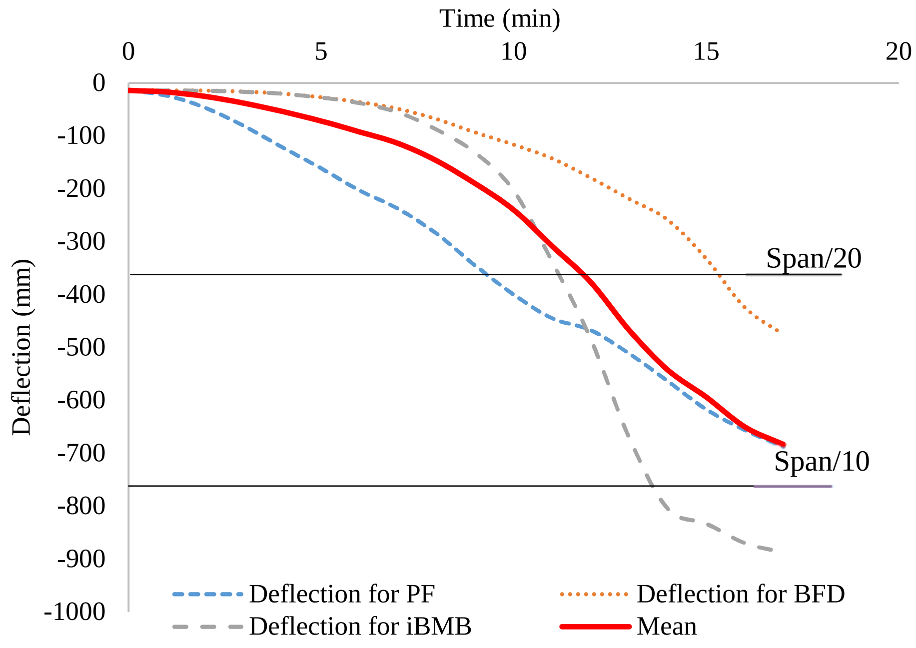


Figure 7.12: Comparison of deflection of 2D beam using mean fires from different fire models.

For the purposes of the 2D analysis, the example used in Chapter 6 was also used here. The composite beam had similar loading conditions as used in Chapter 6, see Figure 6.3.

The composite beam was exposed to mean fires from each fire model and the deflection of the midpoint of the beam was recorded with increasing time step as shown in Figure 7.12. The deflections were compared and the mean of these deflection curves was drawn. The variation of each deflection curve from the mean deflection curve was the uncertainty each fire model brings in to calculate the structural response. It was calculated from Equation 7.9

The deflection curve from the analysis models using the mean fire curve of three fire models was extracted and compared in Figure 7.12. The maximum deviation was termed as an uncertainty in the curve, as shown in Table 7.7.

Table 7.7: Aleatory and Epistemic uncertainty due to fire model

Fire Model	**Uncertainty in fire profiles (β_1)**	**Uncertainty in deflection curve (β_2)**
Parametric Fire	1.00	0.66
BFD	0.44	0.98
iBMB	0.22	0.94

$$\beta = \sqrt{\frac{\sum_{i=1}^{n}[\ln(x_i/\mu)]^2}{n}} \tag{7.9}$$

The comparison from Table 7.7 indicates that the parametric fire produced higher uncertainty when compared to other fire models in generation of fire profiles. This is evident from Figure 7.11 that the parametric fire curve is way below the mean fire curve. The deviation of the structural response curve obtained from analysis using BFD fire model was greater than the other curves as seen in Figure 7.12. As evident in Figure 7.12, the parametric fire model produced high deflection up to span/20 thereafter it started to converge towards the mean curve. The iBMB curve followed the mean curve very closely till span/20 deflection thereafter the deflection increases steeply. The study produced useful results for the failure probability calculation. It is assumed, especially during standard fire testing that structural elements fail at span/20 in order to avoid collapse of structural element setup into the furnace. The epistemic uncertainty due to fire models provided the information that use of any fire model may alter the results. Therefore in calculation this deviation needs to be accounted for while addressing the failure probability.

7.4 Conclusion

In this chapter, a survey was performed in Christchurch to gather information about compartment geometry, ventilation and material properties of the surface linings used in office buildings to identify a mean value of each element and their variation. These office buildings were a mixture of university offices and general offices in the city area. A total of 12 buildings with 79 compartments (approx. 800 rooms) were surveyed with total floor area of 51707 m^2. Since window schedules and surface lining material information for all the buildings were not available, only 3 buildings with 28 compartments (approx. 250 rooms) were surveyed for ventilation and surface lining. The survey revealed the new mean value and standard deviation of ventilation (0.103 $m^{1/2}$ and 0.061 $m^{1/2}$), compartment geometry (659 m^2 and 504 m^2), density (1981 kg/m^3 and 90 kg/m^3), specific heat (1110 J/kgK and 32 J/kgK) and thermal conductivity (1.258 W/mK and 0.095 W/mK) for office buildings in Christchurch.

A new mean value and distribution type of fuel load was calculated for office buildings in New Zealand by combining several existing surveys of New Zealand. This provided the mean value of 805 MJ/m^2 and standard deviation of 424 MJ/m^2. The mean value is very close to the current standard mean value used in NZ as mentioned in Verification Method C/VM2.

On the other hand, this chapter also evaluated the epistemic uncertainty fire models bring in during the calculation of the structural response. It is concluded that a parametric fire produces higher uncertainty in the generation of fire profiles. BFD curves produces higher uncertainty in the calculation of structural response. These uncertainties need to be incorporated during the failure probability calculation. It was found that for the smaller deflection scenarios (<200mm) iBMB has less uncertainty in it whereas parametric fire has the highest for composite beam.

8 CONCLUSIONS

Probabilistic structural fire engineering is an emerging area in structural fire engineering and it is gradually evolving in various ways, such as probabilistic risk analysis, reliability analysis and performance based structural fire engineering. Probabilistic structural fire engineering (PSFE) was introduced to overcome the limitations of current conventional design approaches used for the design of fire-exposed structures, which investigate assumed worst-case fire scenarios and include multiple thermal and structural analyses. PSFE permits buildings to be designed in relation to a level of life safety or economic loss that may occur in potential fire events with the help of a probabilistic approach. It has been noted that although PEER's PBEE framework does apply to PSFE in a broad sense it requires significant attention to properly quantify each stage of the process. This research has redefined PBEE terminologies to suit structural fire engineering. The present work provides a comprehensive review dedicated to the application of the probabilistic approach in structural fire engineering and has evaluated the uncertainties involved in key elements of post-flashover compartment fire and fire models.

The first stage of the PSFE framework is hazard analysis, where fire hazard is represented by a fire severity measure. The collapse of structures occurs when fires are severe. Various elements play an important role in the development of severe fires. These include fire-fighting measures, the effect of active control measures, fuel load, ventilation, the occupancy type and size of the compartment. Fires vary depending on room sizes, their layout, lining materials and the amount of combustible material in the room. There are other sources of uncertainty, such as fire models, structural analysis models, structural configurations, thermal analysis approach, material uncertainty and many more. All these uncertainties have an impact on the evaluation of the probable structural response. Current fire engineering design predicts structural response by a few parameters such as member strength (i.e. force and capacity) and deflection. This reveals that there are more uncertain elements in fire engineering discipline than structural engineering. Therefore, the uncertainty in the hazard analysis stage is far more important than the structural analysis stage. Structural response relates to the damage of the structure, which largely depends on the degree of uncertainty that carries on in the analysis. Unlike PBEE, fire damage measures are either repair or replace. Similarly, at the final stage of PSFE, the decision variables are simpler: repair cost or repair time. The flow of uncertainty through PSFE can be likened to a reverse pyramid, which has more variables or uncertainty at the top and its propagation reduces as the process progresses. This indicates that although PSFE is similar to

PBEE, it does not need to follow the exact footprint of PBEE. The modifications in PSFE were proposed and the implementation of first two stages of PSFE, i.e. Hazard analysis and Response analysis was performed. The conclusion of the studies on the development of PSFE are discussed in succeeding sections and recommendations on future work are made at the end of the chapter.

8.1 Conclusion of Probabilistic Structural Fire Engineering studies:

This thesis has demonstrated the suitability of the adoption of the PBEE approach in structural fire engineering. The study has also investigated various possible FSMs and demonstrated a process to identify the most efficient and sufficient FSM by introducing new selection criteria. New analysis methods have been introduced to avoid extensive scaling of fire profiles as is currently associated with IFA to establish the relationship between FSMs and EDPs. The efficacy of the analysis methods is also evaluated. The research also estimated the annual rate of exceedance of hazard intensity and structural response for New Zealand and Christchurch city with the help of the probability of occurrence of a "structure fire" and the probability of exceedance of a given level of FSM given an EDP. A case study of a composite beam from a typical NZ office building was used to demonstrate the process of selection of an efficient and sufficient FSM and the application of the new analysis methods. The major conclusions were:

- Modifications were proposed to the adaptation of the PBEE framework for PSFE.
- A new method called Fire Stripe Analysis (FSA) was introduced along with Cloud Analysis. Two techniques for the selection of bands (or FSM interval) were discussed in the thesis; 'equal intervals' and 'equal data points'. It is concluded that an "equal data point" approach produces higher dispersion and therefore is not recommended.
- Many FSMs were investigated in PSFE and a new selection criterion, i.e. sufficiency, was introduced. It was found that for a protected composite beam cumulative incident radiation was the most efficient and sufficient fire severity measure for both maximum vertical displacement and maximum steel temperature.
- FSA produced higher dispersions of structural response than Cloud analysis due to the categorisation of FSMs into bands and evaluating the mean and dispersion at each level. There are fewer data points at low FSM level and high level as compared to the intermediate FSM level which produces higher dispersion of response in these ranges.

- FSA produced higher probability of exceedance of structural response at lower CIR due to greater dispersion and because of fewer data points at higher CIR values, it resulted in lower probability of EDP.

- It was concluded that since cloud analysis is a direct approach which does not involve any scaling and uses raw fire hazard data for the analysis, it therefore produced more accurate results, for a limited range where a linear assumption between FSM and EDP stays valid, when compared with FSA. FSA was a unique approach in PSFE and will be very useful where fire profiles are falling in a small FSM bands.

- The comparison of results for unprotected and protected composite beams revealed information about the advantage of fire protection in limiting structural failure. The information is very useful to designers to estimate the amount of additional safety induced in the structure (composite structure) due to fire protection. The graphs of protected and unprotected beams readily show the advantage of the fire protection and provides numeric information.

The annual rate of exceedance of damage and repair cost/time incurred to the structure during its lifetime is another important outcome of the PSFE approach. The probability of failure or the probability of exceedance of fire severity and structural response varies tremendously due to the uncertain nature of fire development factors, fire models, thermal analysis models, structural analysis models and many more. Therefore, this research calculates the mean annual rate of exceeding any particular value of FSM and structural response in Christchurch and New Zealand to put the probability of occurrence of an event into perspective.

- It was observed that Christchurch city has 15% less probability of exceedance of the specified fire severity level than the whole of New Zealand.

8.2 Conclusion of Effect of modelling on failure studies:

In a separate study in this research, the effect of the structural finite element modelling uncertainty on the structural response was evaluated. The deemed motive of this comparison was to investigate the effect of structural configuration on the probability of failure or structural response. This was demonstrated through a study performed on an office building. The response of a beam was observed in two different structural configurations; first as an isolated beam modelled in 2D and then as part of a larger structure in a 3D model, all exposed to fire.

- It was observed that 3D modelling produces less deflections and comparatively less probability of exceedance of a level of displacement.

- It was also important to consider the computation time when selecting the structural configuration for modelling. 3D analysis may produce more accurate results but could be time-consuming.

- It could be concluded that if a member is designed isolated then it will produce conservative results.

8.3 Conclusion of Uncertainty in structural response:

Aleatory and Epistemic uncertainty were evaluated in this research. The randomness in fuel load, ventilation, surface lining and compartment geometry were considered as a variable and a new mean value and distribution type was calculated. The effect of fire model in the structural response was also evaluated in this study. A survey performed on 12 buildings with 79 compartments (approx. 800 rooms) in Christchurch to calculate the latest mean value and distribution type of compartment geometry, ventilation and material properties of the surface linings used in office buildings. These office buildings were a mixture of university offices and general offices in the city area. The results were:

- The mean value and standard deviation was found to be for ventilation (0.103 $m^{1/2}$ and 0.061 $m^{1/2}$), compartment geometry (659 m^2 and 504 m^2), density (1981 kg/m^3 and 90 kg/m^3), specific heat (1110 J/kgK and 32 J/kgK) and thermal conductivity (1.258 W/mK and 0.095 W/mK), for an office building in Christchurch.

- A new mean value of 805 MJ/m^2, standard deviation of 424 MJ/m^2 and normal distribution type of fuel load was calculated for office buildings in the New Zealand region by combining several surveys.

Epistemic uncertainty due to fire models was calculated for fire profiles and structural response. These uncertainties need to be incorporated during the failure probability calculation. The conclusions were:

- Parametric fire produces higher uncertainty in the generation of fire profiles.

- BFD curve produces higher uncertainty in the calculation of structural response.

8.4 Future Work

Based on the literature review and research performed in this thesis, future work is deemed necessary in the aspects outlined below.

1. The results of the research, annual probability of structural response, can be extended to calculate annual loss or damage of a structure in NZ or Christchurch.

2. The mean annual probability of fire hazard and structural response can also be calculated for other occupancy types.

3. This research is performed on steel-concrete framed structure and can be executed on concrete structures with further extension to damage and loss estimation.

4. Extend the epistemic uncertainty calculation using more fire models and produce more accurate results to evaluate the impact of epistemic uncertainty on structural response.

5. Investigate other selection criteria of FSM selection such as predictability, practicality, computability to produce more robust fire severity measure.

6. Identify EDPs which better correlate with damage states in PSFE.

7. Develop or implement other analysis methods of PBEE into PSFE.

8. Develop fragility functions with ideal FSM and EDPs.

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APPENDIX - A

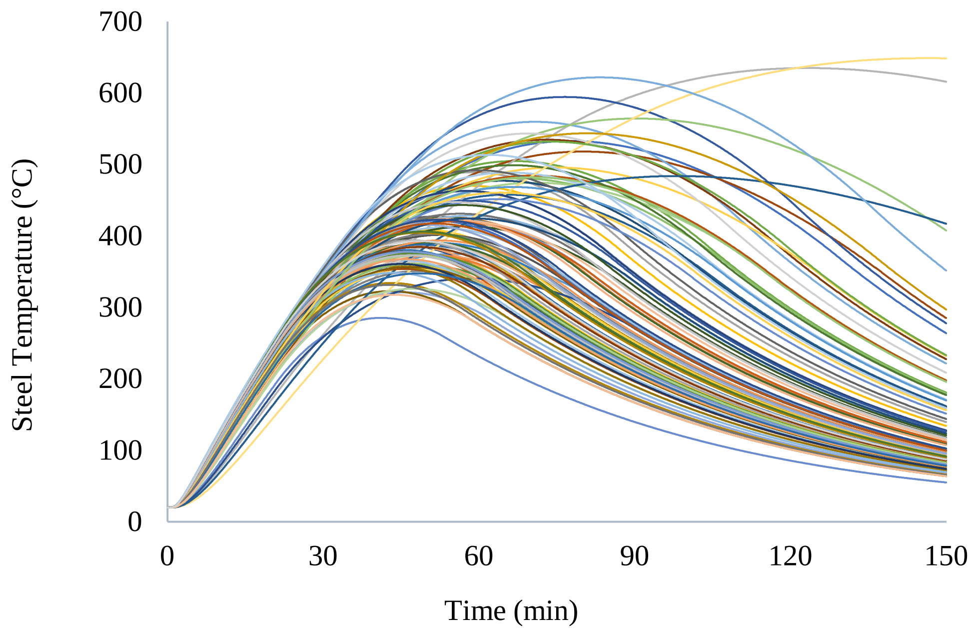

Figure A.1: Steel Temperature profiles

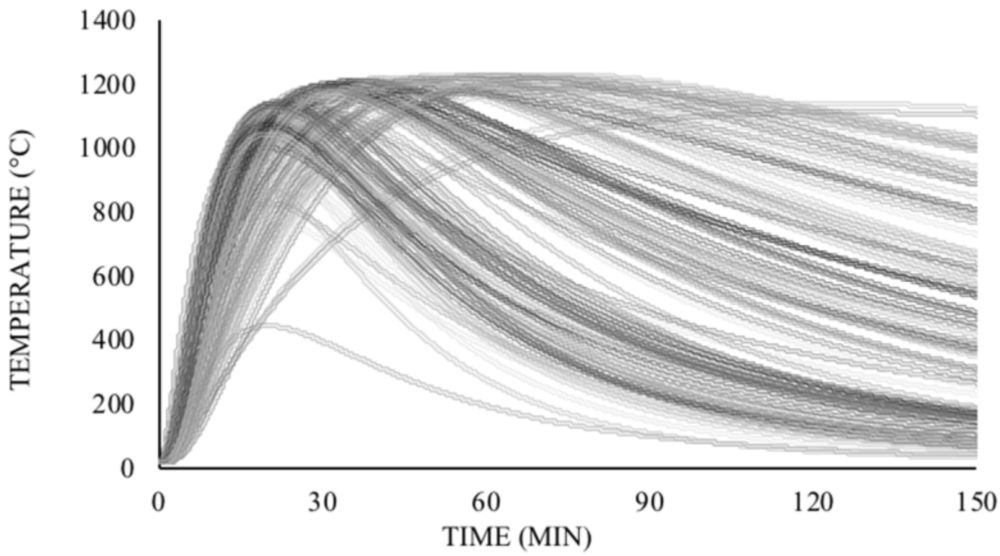

Figure A.2: Temperature-Time profiles from BFD curve fire model

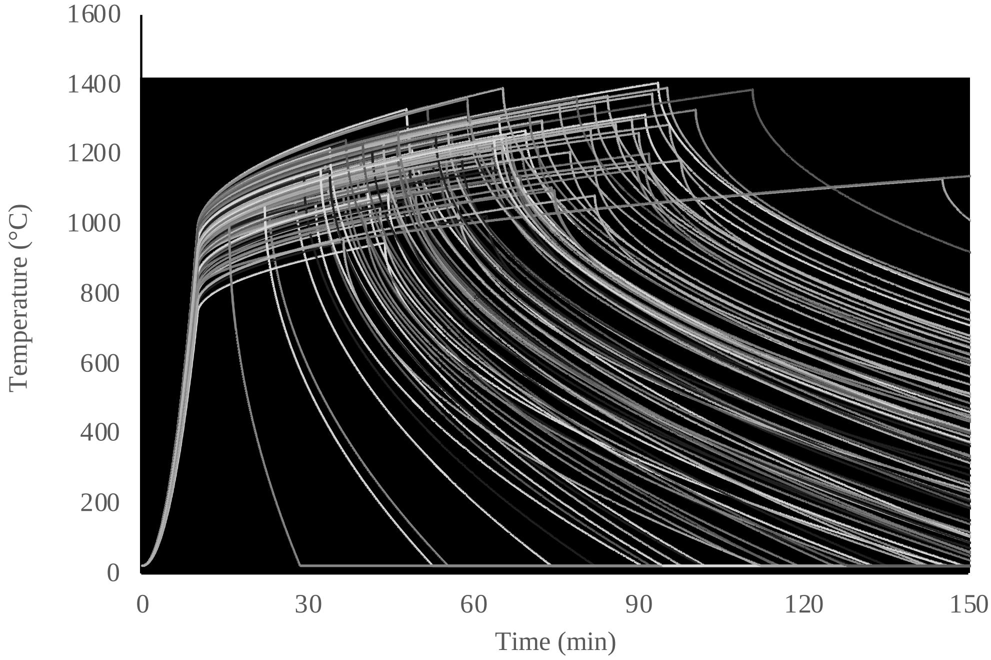

Figure A.3: Temperature-Time profiles from iBMB fire model

Table A1: Collection of Fuel load survey over past 50 years including missing values.

Year	Region	No of Observation	Mean	Standard Deviation	Author
1968	US-1	453	348	104	Bryson/Gross
1957	US-2	82	1270	381	Ingberg
1995	US-3	6	1298	389	Caro
1991	US-4	241	1120	336	Korpela
1986	US-5	419	555	285	Culver CIB W14 report
1986	US-6	625	525	355	Culver CIB W14 report
1995	NZ-1	5	681	226	P Narayanan
2000	NZ-2	7	950	598	H W Yii
1984	NZ-3	1	436	131	Barnett
2008	NZ-4	241	800	240	C/AS1, NZ building
2008	France	61	657	290	Thouvoye
1999	Finland	165	730	219	Korpela
1986	Europe-1	414	420	370	CIB W14 report
1986	Europe-2	241	410	330	CIB W14 report
1986	Europe-3	241	330	400	CIB W14 report
1986	Swedish	241	411	334	CIB W14 report
1993	Swiss-1	241	500	150	CIB 1993 report
1993	Swiss-2	241	600	150	CIB 1993 report
1970	UK-1	93	400	120	Baldwin
1991	UK-2	241	698	209	Butcher
1993	UK-3	801	418	301	Melinek
1993	India	388	348	262	Sunil/Rao
2011	Canada	103	557	286	Ehab/James
1999	AU	241	800	480	FCRC

Table A2: Table showing % error using maximum, minimum and average values in place of missing number of observations.

No of Observation	Combined Mean	Combined SD	% Increase
1	807	457	-
4	805	424	-0.14
7	805	401	-0.24

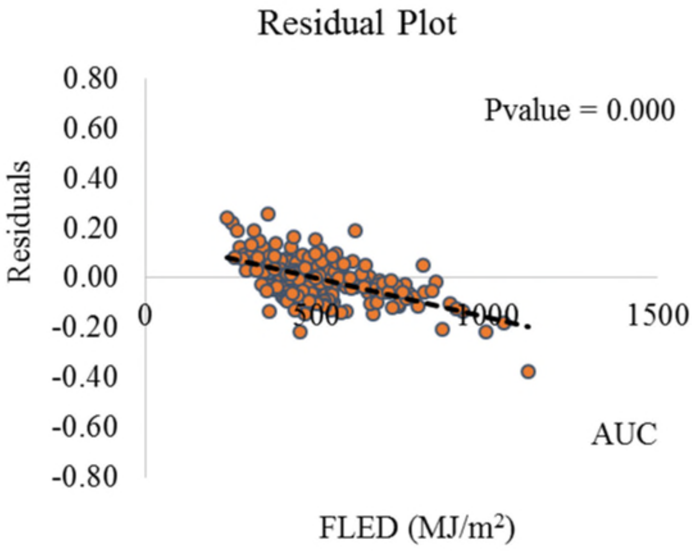

Figure A.3: AUC sufficiency with respect to (i) Fuel load and in predicting maximum displacement of protected composite beam

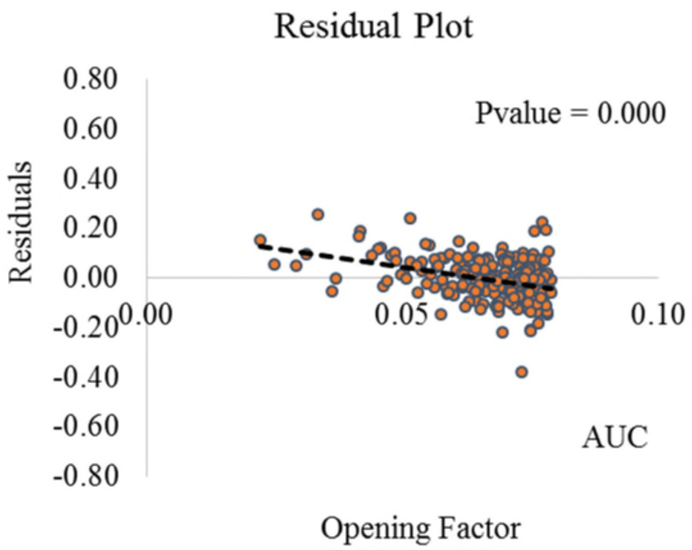

Figure A.4: AUC sufficiency with respect to Opening Factor in predicting maximum displacement of protected composite beam

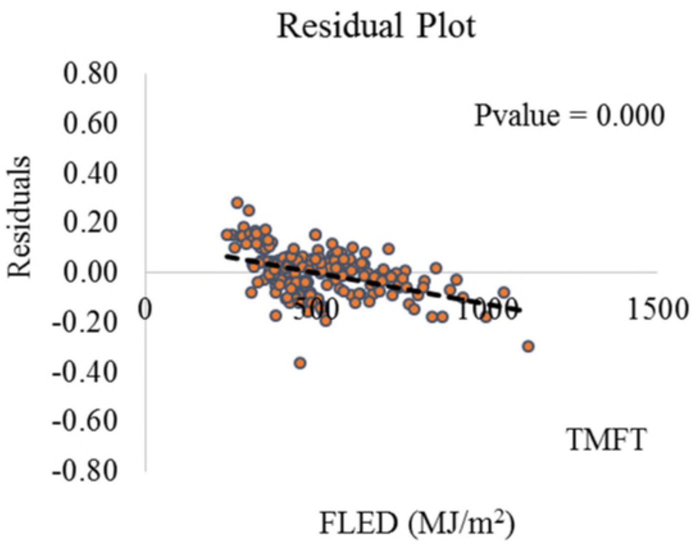

Figure A.5: TMFT sufficiency with respect to Fuel load in predicting maximum displacement of protected composite beam

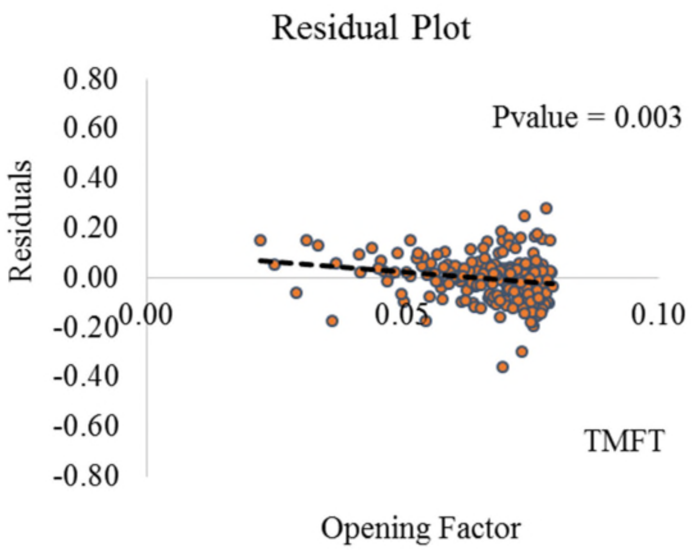

Figure A.6 TMFT sufficiency with respect to Opening factor in predicting maximum displacement of protected composite beam

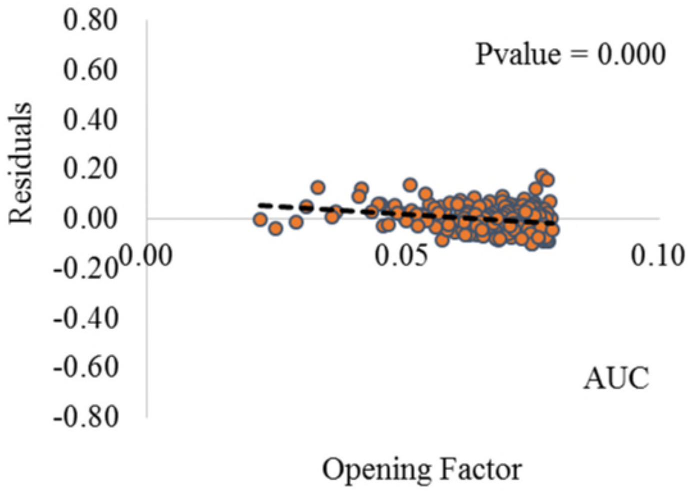

Figure A.7: AUC sufficiency with respect to Opening factor in predicting maximum steel temperature of protected composite beam

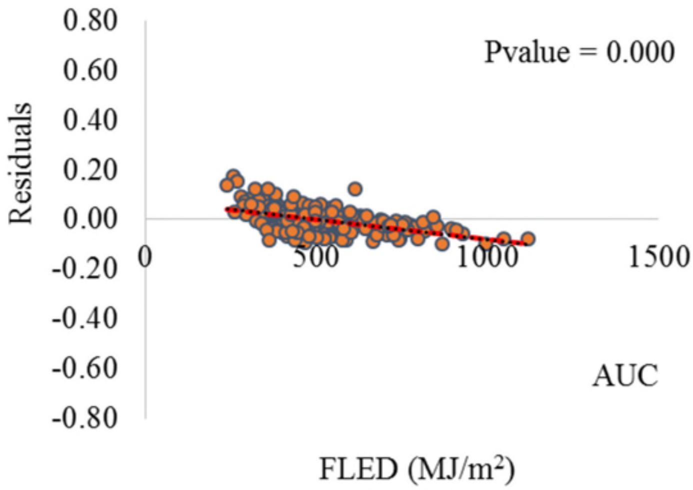

Figure A.8: AUC sufficiency with respect to Fuel load in predicting maximum steel temperature of protected composite beam

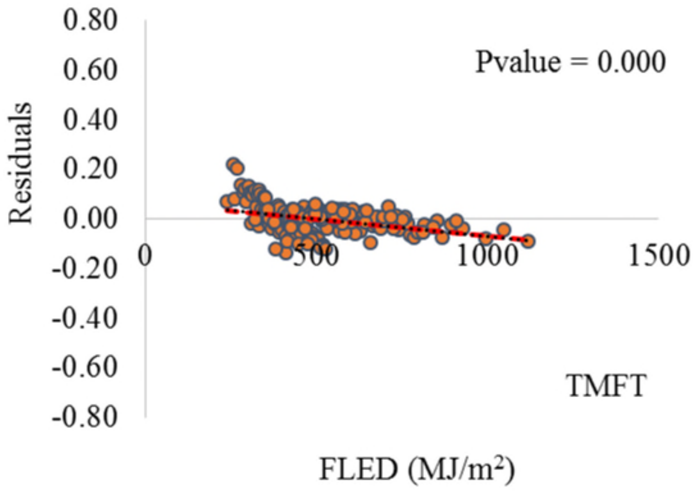

Figure A.9: TMFT sufficiency with respect to Fuel load in predicting maximum steel temperature of protected composite beam

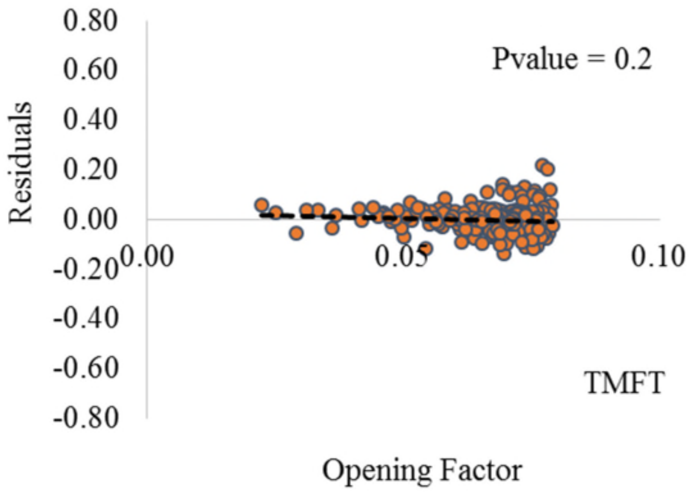

Figure A.10: TMFT sufficiency with respect to Opening factor in predicting maximum steel temperature of protected composite beam